GEOTECHNICAL INVESTIGATION
& GEOHAZARD REPORT
JONES EDGEMOOR ESTATE
PROPOSED 38-LOT PLAT DEVELOPMENT
VIEWCREST ROAD, BELLINGHAM, WA

Submitted to Ann C Jones, Family LP
November 3, 2021

Submitted by:
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November 3, 2021

To: Ann C Jones, Family LP
807 Chuckanut Shore Road, Bellingham, WA 98229

Subject: **Geotechnical Investigation & Geohazard Assessment**
Proposed 38-Lot Plat - Jones Edgemoor Estate
Viewcrest Road, Bellingham, WA

Dear Ms. Jones,

Element Solutions (Element) is pleased to present the following Geotechnical Investigation for the above referenced project and site. This report was compiled using information provided by the project team, desktop review of public information, field reconnaissance with slope observation, subsurface geotechnical explorations, laboratory testing, review and analysis of conditions encountered, and the professional judgment of our geotechnical professionals.

The work plan generally included review of the study area and mapped geologic conditions, field reconnaissance and visual assessment of existing site conditions, and a subsurface investigation that entailed the logging and evaluation of twenty-six (26) exploratory test pits. Reconnaissance for observation of slope conditions, interpretation of geologic hazards, and assessment of exposed bedrock characteristics was performed on several dates during the course of this study. Test pits were observed on June 30 and July 1, 2020, at locations dispersed throughout the upland areas of the site interior as current access allowed. Additional explorations for utility construction planning were completed along Sea Pines Road on November 13, 2020, including two (2) machine test pits and two (2) hand auger borings. Our interpretations and conclusions regarding geologic hazards and subsurface conditions across the study area, based on work completed to date, are summarized in the following report.

This report is intended to provide the project team with site-wide geologic information, project feasibility commentary, and relevant geotechnical recommendations to inform project decisions, conceptual planning, and engineering design considerations for the proposed plat at the Jones-Edgemoor Estate property.

Thank you for the opportunity to work on this project. Should you have any questions regarding this report, please contact us at (360) 671-9172. Element Solutions is a wholly owned subsidiary of Pacific Surveying & Engineering.

Sincerely,

John R Gillaspy, LEG, M.S.
Environmental Services Manager
ELEMENT SOLUTIONS

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1 Introduction

1.1 General Overview

Element has completed this geotechnical investigation and geologic hazard assessment on behalf of the clients, property owners, for contribution to the plat design and approval process for proposed residential development of the project site. In general, the work was conducted to provide a distributed subsurface site characterization and inform preliminary geotechnical aspects of project feasibility planning and engineering, including the influence of steep slopes within and bordering the development area. The project entails establishing a new plat with approximately 38 lots and associated road and utility infrastructure in Bellingham, Washington. The project site is located south of Fairhaven, on Viewcrest Road, in the Edgemoor neighborhood. The site is situated within a hilly and forested upland area bounded by a sheltered bedrock bluff slope defining the northern margin of Chuckanut Bay. Refer to Appendix I (Figures 1 and 2) for maps depicting the general site location, surrounding vicinity conditions, and current proposed lot and road layout.

1.2 Project Understanding

The proposed project involves future plat development of the currently vacant and forested hilly site with a single-family residential community. The project is currently in the design stage and subject to changes in layout at the time of this report. Preliminary layout plans (Pacific Surveying & Engineering) indicate that 38 residential parcels are anticipated to be created within the plat. One open space tract and one reserve tract will also be created within areas largely occupied by wetlands or geologic hazards and their associated buffers.

Two main neighborhood roads are planned to service the site, branching from a single entry at Viewcrest Road, at the north-central end of the project area. The roads are shown to extend immediately southward from the main road, then branch southwest and south across the middle of the site following existing topographic benches traversing between areas of steeper or more variable topography. Both roads will terminate in cul-de-sacs within the site. Several shared access driveways are planned to extend from the sides or ends of the main roads to service each lot.

Current road grading plans indicate the roadway corridors will be prepared using a combination of cuts and fills to address local variations in topography. Commonly, the northwest sides of the roads will involve new cut slopes, while the southeast sides will be constructed at grade or over some extent of structural fill.

No information is available on proposed lot grading or foundations, which will be addressed in later lot-specific designs. Based on standards of practice in the area, we presume the future structures will typically use stepped foundations and/or daylight basements where topography is variable or sloping. No excessive fill placement or unrestrained cuts are anticipated for lot preparations. Structural loads are expected to be typical for the scale of single-family residences with wood framing. No unusually heavy, variable, vibratory, or cyclic loads are anticipated.

The majority of stormwater generated from new impervious areas on roads and lots is expected to be collected, treated as necessary, and routed to upland dispersion areas and/or a main tightline outfall leading down to the shoreline. Stormwater from the northwest portion of the project area may be infiltrated as feasible, and otherwise directed to existing utilities along Viewcrest Road.

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1.3 Purpose and Summary of Scope

The purpose of our investigation was to conduct a feasibility-level geotechnical evaluation and large-scale geologic hazard assessment in support of the proposed plat application and its public road improvements. The scope of work performed was in general accordance with the executed project agreement, with adjustments made during the course of the project based on actual conditions encountered. An additional scope of work was completed upon request in support of utility design along the western terminus of Sea Pines Road.

In summary, our final scope of site investigation has included:

1) Desktop review of existing geologic and soils information for the project area (as based on mapping by others and public information), as well as GIS analysis and imagery review of on-site and proximal off-site sloping topography.
2) Site visit for planning of access, utility notification marking/filing, and verification of utility clearances prior to conducting geotechnical explorations.
3) Direction and observation during excavation of twenty-six (26) test pits within the plat project area by a subcontractor, using a rubber-tracked mini-excavator, to termination depths of 2.0 to 8.0 feet below existing ground surface (bgs).
4) Visual reconnaissance of site interior areas to generally assess the character of slopes, observe for and map geologic hazards, and document/measure exposed bedrock structures.
5) Additional explorations off site at Sea Pines Road for utility construction planning. Two (2) test pit excavations and two (2) hand auger borings were performed at the western end of Sea Pines Road, near the eastern boundary of the project site.
6) Review and analysis of field data to assess targeted infiltration potential, slope stability, and formulate feasibility-level geotechnical recommendations for plat development.

1.4 Assumptions and Limitations

The composition and characteristics of subsurface soils were assessed by the observing geoscience professional using available geologic information and field interpretations at the time of excavation. It is possible that soil conditions, variations, or transitions occur that are not fully characterized or identified by the field observations and sampling/testing program.

No data is available for exploration depths and locations other than those recorded in the attached exploration logs. The composition and physical properties of the substrate below those depths, or in areas beyond the immediate exploration locations, cannot be determined without additional geotechnical evaluation. Soil composition, groundwater depth, and the physical properties of the substrate can vary considerably depending on geographic location, elevation, and seasonal or climactic factors. Such variability should be expected and anticipated over the study area. The actual character and type of bedrock may also vary among areas between rock exposures.

Groundwater conditions are likely to vary seasonally, and may also differ between locations within the site. The reported groundwater conditions are valid only for the date and location of exploration. If necessary for design, additional targeted explorations or seasonal monitoring of groundwater should be completed.

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2 Desktop Review and Interpretation

2.1 Methods

The following desktop analysis was conducted by a qualified earth science professional and, although it is built on previous studies and information obtained by others, it includes new interpretations based on professional judgment and experience. The desktop data inventoried in Table 1 cites the available geospatial data for the subject area, which was evaluated using scientific methods based upon industry best practices.

Table 1: Data Used for Desktop Analysis

Data	Format	Date	Source
Aerial photography (Orthophoto)	SID/JPG	2017/2019	USDA/Whatcom County
LiDAR	Bare earth grid	2017	NPSL
Geology	Shapefile	2006	DNR 1:100,000 Digital Geology
Soils	Shapefile	Current	USDA/NRCS Soil Survey

2.2 Location and Physiography

The large-acreage site is located within the southwestern-most part of the City of Bellingham, on the northwest end of Mud Bay. The main site frontage is along Viewcrest Road in the Edgemoor neighborhood of Fairhaven. The site is on the south side of the road, and extends south and downhill to the bay shoreline. The east margin of the site runs north-south near the cul-de-sac terminus of Sea Pines Road. The west margin runs north-south near the cul-de-sacs off South Clarkwood Drive. Bordering sites to the north, east, and west are predominantly developed and in present use as single family residential properties with similar scales of buildings and exterior improvements as the proposed project development. Refer to Appendix I (Figures 1 and 2) for maps depicting the general site location, project boundaries, and surrounding vicinity conditions.

The property is comprised of several contiguous parcels totaling 37.4 acres. The site interior remains generally well forested, populated with mixed conifers and deciduous trees of varying ages along with mature typical undergrowth (ferns, small brush). The site exhibits variable, hilly upland topography throughout a majority of its land area. The upland topography is similar in character to that of residentially developed areas to the east and west. The area along the Viewcrest Road frontage is very gentle to flat, and cleared in the northeastern region of the site while remaining forested in the northwest area. The southeast portion of the site, well outside of the plat development area, consists of a large shoreline bluff slope, over 40% grade and around 100 feet in height, extending down to the shoreline. Further review of slopes within the proposed project development area is provided below.

2.3 Geologic Background

The early geologic history of the northern Puget Lowlands is defined by tectonostratigraphic terrane accretion. Volcanic island arcs and associated terrestrial and marine sedimentary units collided with and were incorporated into the continental margin during subduction of the oceanic Farallon plate. This process was ongoing through the upper Mesozoic Era and resulted in the highly faulted and deformed exotic terranes associated with the exhumed and uplifted Northwest Cascades System.

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By the lower Cenozoic Era, the crustal material comprising basement rock of the Puget Lowland had formed a pull-apart basin submerged beneath a shallow subtropical sea, which received both continental and marine sediment inputs. This depositional period, constrained to roughly 58 to 50 MA (Lapen, 2000), resulted in the thick sandstone, conglomerate, mudstone, siltstone, and bituminous to subbituminous coal of the Chuckanut Formation prevalent in the Bellingham area. Later folding, tilting, and uplift of the sedimentary unit caused the complex bedding patterns that influence and are exposed by today's landscape. Various continental glacial episodes occurred in recent geologic history, capping valleys and low coastal areas with thick glacial sediments, and commonly mantling foothill areas with thin glacial drift or till soils. Among hilly lowland areas such as the project site, it is common to see a range of shallow conditions over bedrock at depth. Shallow soils can include bedrock-derived colluvium, glacial drift/till, glacial outwash, and locally fine alluvial or organic deposits.

Geologic mapping at 1:100,000-scale, conducted by the Washington Department of Natural Resources (DNR), indicates that the study area is underlain by the Padden Member of the Chuckanut Formation (Ec_{cp}). The Padden Member is a sedimentary bedrock unit described as "moderately to well-sorted sandstone and conglomerate alternating with mudstone and minor coal. Sandstone ranges from fine to coarse grained, with pebbly to conglomeratic sandstone layers common" (Lapen, 2000). In our experience, it is common for bedrock to be overlain by about 2 to 5 feet of cover soils such as colluvium or mantling glacial deposits, varying locally.

2.3.1 NRCS Web Soil Survey

The United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Web Soil Survey for Whatcom County indicates that there are two primary soil units in the study area; *Everett-Urban Land Complex, 5 - 20 percent slopes* (NRCS Map Unit 52) extending into some northern areas of the site, and *Nati loam, 30 - 60 percent slopes* (NRCS Map Unit 110) across the central and southern majority of the site interior.

Everett-Urban Land Complex, 5 - 20 percent slopes (NRCS Map Unit 52)
This unit typically forms on moraines and terraces from a parent material of loess and volcanic ash over glacial outwash. Typical soil profile consists of gravelly ashy sandy loam through 25 inches depth, then very gravelly sandy loam, loamy sand, and sand through 60 inches depth. The Everett soil is somewhat excessively drained, but has a very low to low capacity to transmit water through its most limiting layer. The unit is assigned Hydrologic Soil Group B and not noted as being prone to flooding or ponding. Depth to seasonal groundwater is typically between 39 to 59 inches. Restrictive flow conditions (densic material) is encountered in the range of 40 to 60 inches depth.

Nati loam, 30 - 60 percent slopes (NRCS Map Unit 110)
This unit typically forms on hillslopes from a parent material of volcanic ash, colluvium, and slope alluvium derived from sandstone, siltstone, and glacial drift. Typical soil profile consists of ashy loam through 38 inches depth followed by weathered bedrock to 42 inches depth. The Nati soil is well drained and has a moderate to high capacity to transmit water. The unit is assigned Hydrologic Soil Group C and is not noted as being prone to flooding or ponding. Depth to seasonal groundwater is typically greater than 80 inches. Paralithic bedrock is typically found beginning in the range of 20 to 40 inches depth.

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The findings of our explorations are broadly consistent with the geologic and soil survey mapped units. The shallow soil column consists generally of glacial drift or colluvium and is capped with thin cover deposits derived from or composed of weathered native materials. Drift and colluvium deposits are underlain by bedrock consistent in composition and character with the regional Chuckanut Formation. Exposures on steep rock outcrops are also consistent with the folded sedimentary layering of the Chuckanut Formation.

2.4 Geologic Hazard Commentary

Due to the prevalent and variable sloping grades within the project site, and its bordering slope conditions, we performed an initial image review of topography and slope characteristics to determine the approach and focus for reconnaissance-level field review. In the course of this study, we assessed the presence of any obvious active geohazard features, as well as to determine if on-site or proximal areas fall under standard critical area designations for steep slopes as defined by gradient. City of Bellingham Municipal Code (BMC) 16.55.420(B) defines *Landslide Hazard Areas* (LHAs) as slopes having a consistent grade of 40% or greater and a height change of at least 10 feet. *Erosion Hazard Areas* (EHAs) are defined as areas of topography exceeding 30% which are underlain by erosion-prone soil types. BMC language does not differentiate between areas of steep grade and areas indicating active or historical instability; however, this is an important designation for assessing stability and risk of future hazards. For the purposes of our review, we refer to *potential* LHAs as areas of steep grade (over 40% and 10+ feet height), versus *active* or *historical* LHAs defined by interpretation of presence where applicable.

2.4.1 Slope Gradient Review

City of Bellingham CityIQ GIS data (accessed on-line) was initially reviewed for topographic information relating to slope grades. Within the subject site, slope grades are shown by this resource to vary typically under or over 15%, with some prominent hills and scattered features over 40% sustained grade. Steep slopes of the site development area are shown as under 100% grade (1:1), with exception of a steep rock exposure in the northwest quadrant. The regularity of slope occurrence prompted our further detailed spatial analysis using LiDAR-based topography.

The results of our detailed GIS-based topographic analysis are shown in Figures 3a and 3b. This detailed approach demonstrates that a majority of the site interior has grades of under 15% or between 15% and 30% (not regulated by critical area code), shown in light green and yellow shading, respectively. Small scattered areas within otherwise gentle topography are shown as exceeding 30% grade; however, the isolated occurrences are likely to reflect small surface variations on the scale of a few feet that are not indicative or relevant to development regulation. We conclude that the site generally <u>does not</u> contain EHAs that are not associated with and more appropriately classified as LHA areas (either *potential* or *identified*).

Areas over 40% grade are shown on Figures 3a and 3b as orange, and grades over 80% in red. ***The site contains various slope features within the development area that are correctly classified as potential LHAs***. The steepest grades within the project area occur on the southeast faces of hillsides, and generally correspond to areas of bedrock exposure. Some isolated and small but steep features appear to be related to historical primitive road cuts. In Section 4, we present the findings of visual field review of steep slopes and steep rock exposures; and we provide interpretations of site stability based on a combination of reconnaissance findings and field data.

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2.4.2 Special Hazard Areas

Two features of special significance were evident in initial image review. These include: 1) the main southeast shoreline bluff slope, and 2) an area of bowl-shaped topography at the northeast corner of the property. Figure 4 presents site-wide LiDAR imagery including delineation and annotation of the areas noted below.

1) The southeastern bluff slope is consistently steep, commonly over 80% grade, and in some areas exceeding 100% grade (1H:1V). The crest of the southeast slope is at roughly 80 feet to 120 feet in elevation (above sea level) depending on area. The crest typically exhibits an over-steepened top of the slope and shows signs of past localized mass wasting activity (serrated trend with cuspate features). The main body of the slope face varies between around 40% and over 80% grade with an overall slope of about 1.5H:1V. From aerial and shoreline photography, we can see that the slope and upland area behind the crest remains forested with mature evergreen trees. At the base of the slope and along the face, are visible areas of exposed bedrock that appear to be dipping moderately or steeply northward into the hillside. We interpret the slope to be comprised of intermittent outcrops of steep resistant bedrock planes, interspersed with colluvium slopes that are reclined enough to support the existing forest vegetation. Despite the locally hazardous features present, we infer that the slope has a high degree of internal global stability as a function of the bedrock-structure orientation. The plat development proposes an "open space" tract along the entirety of this feature. Furthermore, the lots proposed uphill from its crest are sufficiently large to permit a substantial setback (well in excess of 100 feet) from the bluff slope. In our opinion, a detailed review of the feature is not necessary for plat approval.

2) The northeast corner of the project area, to the west and northwest of the Sea Pines Road terminus, exhibits geomorphic features indicative of a historical landslide feature (Figures 4 & 5). However, the actual history of the feature is not known. Signs of potential historical mass wasting activity include a concave and convergent topography, arcuate slope crest, and steeper upper scarp with lower-angle interior slope. The presence of wetlands within the interior basin is also consistent with this interpretation. With exception of its northernmost areas downhill from other off-site residences (lots not in project area), the crest is somewhat diffuse below and adjacent to the project development area, indicating some time since formation of the landform. We infer that this is likely a historical mass wasting feature with local crest reactivation or episodic retreat occurrences at its north end. The likely cause(s) of the feature at its location are not clear. It is plausible that the area originally held thicker soil deposits than elsewhere, and may have been influenced by concentrated runoff, or subsurface groundwater concentration (given the wetland presence). It is also possible that the feature originally dates back to the time of late-stage glacial recession, when surface conditions were more volatile. We have delineated the approximate boundaries of the feature (Figures 4 & 5), and the preliminary plat layout has been adjusted for avoidance of its extent plus standard 50-foot landslide hazard buffer. Based on the avoidance, no further review is necessary.

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3 Geotechnical Explorations

3.1 Methods

Site surface characteristics within the project area were evaluated in the field during reconnaissance by the geotechnical team prior to and at the time of the field explorations. A total of twenty-six (26) test pits were completed, on June 30 and July 1, 2020, to directly observe and evaluate the subsurface conditions throughout the interior of the project site. Test pits were excavated by a subcontractor, using a Yanmar EX35-5 mini excavator, to termination depths ranging from 2.0 feet to 8.0 feet below the existing ground surface (bgs). Exploration locations were selected based on access and to provide optimal representative coverage of the site as conditions allowed. Test pit locations are indicated on Figure 6, Appendix II. Detailed exploration logs and laboratory testing reports are also attached in Appendix II. Select photos of representative conditions observed in test pit excavations are shown in Exhibit A.

3.1.1 Subsurface Investigation

Twenty-six (26) test pits were excavated at representative areas within the project site as access allowed at the time of the work. General exploration areas were pre-selected by Element geotechnical staff based on the provided preliminary development plan, and field-located by an Element Solutions geologist during initial site reconnaissance. Final test pit locations were adjusted based on existing access and utility considerations. Each test pit and boring location was marked in the field using a hand-held TOPCON FC-5000 GPS unit (±3 m accuracy).

Soils observed during explorations were classified by visual means according to the ASTM D2488 Soil Engineering Classification System. Subsurface water and high moisture conditions, including apparent groundwater level, seepage occurrences, and saturated soils, were also noted as encountered during explorations.

An Element geologist collected representative direct grab samples of soils encountered in test pit excavations. Samples were placed in sealed plastic bags for transport and storage. Following field activities, samples were re-examined to confirm field classifications. Representative soil samples were then submitted for laboratory testing to aid in final classification and for use in analysis of soil design properties. Remaining samples will be stored temporarily by Element; additional testing of samples can be conducted at request of the client.

3.2 Subsurface Soil Conditions

Subsurface soil and bedrock conditions encountered in the explorations were broadly consistent with regional geologic and soil mapping. The explorations support the overall geologic interpretation of the site as underlain by shallow bedrock and associated cover deposits; capped or mantled by glacial outwash, glacial drift, and glacial till varying locally. Cover soils thickness and character differed by location, but generally consisted of organic-rich topsoil underlain by silty sand of glacial deposition or rock-derived origins.

A brief summary of the observed soil horizons is presented below. For complete information, refer to the attached exploration logs (Appendix II). The interpreted geologic unit for each horizon, corresponding to the summaries below, is shown in bold with the soil description.

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Uncontrolled Fill: Shallow materials, interpreted as non-native uncontrolled fill were found at one location (TP1, northeastern margin area) to approximately 3.5 feet bgs. The location coincides with an area of somewhat raised grade at the northern extent of the "East Road", currently a primitive and overgrown off-road feature. Based on topographic indications, we suspect that similar fills may extend into the properties located to the east and west of TP1. The fill consisted of silt with sand (USCS Classification: ML) containing approximately 50% to 60% fines, was soft to medium stiff with depth, cohesive with low plasticity, and damp in the early summer season. The fill contained some chunks of asphalt, and was capped with about 0.7 feet of topsoil. A band of dark orange oxidation staining was observed from about 3.0 to 3.5 feet bgs near the base of the fill material.

Topsoil: Organic-rich silty topsoil (USCS Classification: OL) was present at the surface of all exploration locations to depths ranging from approximately 0.3 feet to 3.0 feet. With the exception of TP3, topsoil horizons found in test pits along the primitive northeast-southwest (NE-SW) trending access corridor (TP2 to TP12 run) were all less than 0.9 feet thick and had an average thickness of about 0.5 feet. The limited depth may be due to prior partial stripping. The northwest margin of the site exhibited a more well-developed and thicker topsoil horizon, often in the range of 1.5 feet to 3 feet. The organic silt displayed consistent characteristics throughout the study area, and contained occasional cobbles and root material. The topsoil was generally dark brown to medium reddish-orange brown, soft, and damp to moist.

Glacial Deposits:

Glacial Drift

Interpreted glacial drift deposits encountered on site were composed of predominately coarse-grained material containing varying degrees of fines, gravel, cobbles, and occasional boulders. Glacial drift soils along the primitive NE-SW access corridor were predominately comprised of silty sand with some gravel and cobbles (USCS Classification: SM) and fine fractions in the range of 20% to 40%. The SM soil was commonly gray to grayish brown, non-plastic, low to moderately cohesive, and typically medium dense at shallow levels before transitioning to dense glacial till or bedrock conditions below. Gravel clasts were sub-rounded to rounded, as were the occasional boulders observed within the unit. Soil water content was generally noted as damp to moist conditions and decreased with depth. Mottling and oxidation staining was often observed in the drift soils, decreasing or vanishing with depth into basal till or unweathered bedrock.

Glacial Outwash

A soil horizon ranging between 1.2 feet and 3.0 feet thick, interpreted as glacial outwash (recessional), was uncovered below the topsoil in the northwest area of the site (TP13 - TP17). The outwash soils were composed of a variety of well- to poorly-graded sand and gravel, with some cobbles, and fine silt content ranging from about 2% to 20%. The granular soils were medium dense, non-cohesive, non-plastic, and damp to moist. Coloring was grayish brown to light gray in test pits where sand was the dominant constituent; and brown to orange brown in areas dominated by gravel. Clasts were rounded to well-rounded, and some caving was observed in test pit walls. Other than TP15, where refusal was met on a large boulder, dense glacial till was found at the base of outwash soils. Outwash-type soils were observed to overlie Drift soils at multiple test pits, and elsewhere was found in substitution for Drift deposits.

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Glacial Till

A medium dense to densely compacted mantle of glacial till was found overlying bedrock at a majority of test pits (excluding locations on or near the tops of outcrops). The till unit was composed of grayish brown to light gray silty sand containing some clay, gravel, and occasional cobbles (USCS Classification: SM). Fines content was generally in the range of 20% - 40%, sand content was medium to fine-grained, and gravel clasts were often small and rounded. The SM soil displayed low to moderate cohesion and low plasticity. The density of the till increased greatly in the last 0.5 feet to 1.0 feet of the unit, becoming cemented and blocky, often forming a thin veneer over the underlying bedrock. The upper horizon of the till was locally-weathered and weakened, but became progressively dense with depth. Moisture content was generally low and decreased with depth in concert with an increase in densic or cemented and blocky texture.

Colluvium: Soils distinct from glacial deposits and interpreted as derived from on-site bedrock, either redeposited (colluvium) or weathered in place (regolith / paralithic rock), were observed in areas throughout the site; most often in test pits located on slopes or in high elevation areas. The rock-derived soils were generally comprised of tan to yellowish brown silty sand with some gravel and cobbles (USCS Classification: SM) containing approximately 20% to 30% fines content. Sand was poorly graded and mostly fine to medium. Gravel and cobble clasts were tan and angular. The SM soil was damp, non-plastic, displayed low cohesion, and was medium dense to dense as it transitioned into the more intact weathering rind of the underlying bedrock. At multiple locations in the north-central area of the site (TP-18 & 19), this deposit was found underlying Glacial Drift. Due to the nature of colluvium deposits, they may range in age and character by location.

Soils that appeared to have been weathered-in-place (eluvium) were observed at the top of the outcrop in the northwest region of the site (TP25 and TP26). These soils appeared similar in character to the more frequently observed colluvium, but were made up almost entirely of poorly-graded medium sand (USCS Classification: SP), containing less than 5% fines. The SP soil was yellowish brown, non-plastic, non-cohesive, damp to moist, and loose to medium dense in the upper 3.5 feet before transitioning to the underlying weathering rind and bedrock at 4.0 feet bgs.

Bedrock: Apparent intact sandstone bedrock of the Chuckanut formation was encountered in a majority of test pits across the study area. In the southeast part of the project area, along the primitive NE-SW access corridor (TP2 - TP12), the depth to bedrock was consistently less than 4.5 feet, with exception of TP8 where bedrock was encountered at 8.0 feet bgs. The depth to bedrock was only slightly greater along the proposed "West Road" corridor and in the central region of the site, where refusal was generally met at around 5.0 feet bgs or less. Extracted rock samples were comprised of angular, dry, tan, poorly-graded sand to silty sand. The inferred bedrock conditions are consistent with the Padden Member of the Chuckanut Formation, mapped in and around the study area and exposed in scattered outcrops. See Figure 7 for a summary of depth to bedrock by exploration location.

Bedrock was not encountered in the northwestern corner of the site at the TP13 - TP15 locations, which were terminated in dense till-like conditions or on a large boulder. This suggests that depth to bedrock is greater in the northwest corner of the site. It is also common for the Chuckanut Formation rock profile to vary locally. The depth to rock encountered along the primitive access corridor and proposed "West Road" alignment was relatively consistent and may be broadly representative of the site. However, as observed at TP8, local variation should be expected.

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3.2.1 Laboratory Testing Results

Grab samples were collected from test pit excavations at the depths noted on the logs. Following field work, we reviewed the exploration findings and selected representative samples for laboratory analysis to confirm soil properties and visual classifications. Samples were delivered to GeoTest Services, Inc. for hydrometer analysis (ASTM D422/D1140 method), sieve analysis (ASTM C136/C117 method), percent passing #200 (fines content), and Atterberg Limits (Plasticity Index) testing. Organic content (ASTM D2974 method) and cation exchange capacity (EPA 9081 method) testing were performed by Northwest Agricultural Consultants. The sample array and test results are indicated in Table 2 below. Complete laboratory test reports are attached in Appendix II.

Table 2: Summary of Laboratory Testing Results[1]

Sample ID	% Gravel		% Sand			% Fines		Atterberg Limits			USCS
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay	LL	PL	PI	
TP1 - 6’	8	15	5	11	30	20	11				SM
TP2 - 2’	0	8	4	23	54	11					SP-SM
TP8 - 4’						20		51	25	26	SC
TP9 - 4’						22					SM
TP10 - 3’	0	5	2	17	55	21					SM
TP12 - 3’						28					SC/SM
TP13 - 4’	28	28	12	27	3	2					GP
TP13 - 6’	0	20	8	21	23	21	7				SM
TP16 - 3’	26	26	9	22	12	5					GP-GM
TP16 - 4.5’						39		21	16	5	SC-SM
TP24 - 4’	19	21	10	21	21	6	2				SP-SM
TP25 - 2.5’						2					SP

1. Test results from Northwest Agricultural Consultants:
   a. TP1 (6.0’): Organic Matter = 1.77%; Cation Exchange Capacity = 11.6 meq/100g
   b. TP13 (4.0’): Organic Matter = 1.50%; Cation Exchange Capacity = 3.9 meq/100g
   c. TP24 (4.0’): Organic Matter = 1.44%; Cation Exchange Capacity = 6.2 meq/100g

Gradation results from all samples indicate that fines content of the glacial deposits ranges from as low as 2% to as high as around 40%, with typical values between 20% to 30% fines in the drift and till soils and below 10% to 20% for the local outwash deposits. Field assessment of soil plasticity suggested non-plastic to low plasticity behavior in a majority of observed soil types. Atterberg Limits testing of two fine soil samples recorded plasticity index (PI) values ranging from the region of low plasticity silty clay (CL-ML) up to the lower limit of high plasticity clay. Given the depositional source and our field observations of soil character, some variation of fine and coarse fractions and range of plasticity (from non-plastic to low plasticity) is expected, with most soils behaving as non-plastic or low plasticity.

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3.3 Groundwater Conditions

Weather conditions were mostly dry during field work with only minor precipitation occurring during the first day of explorations. No excessive surface ponding was observed during field reconnaissance or explorations, outside of designated wetland areas (not assessed in this study). Groundwater and free water conditions were observed directly in excavations. Soils were generally damp to moist throughout the study area. Wet soils were only seen in TP2, where seepage and caving were observed at a depth of 3 feet bgs. However, heavy oxidation staining indicated groundwater levels rise to around 2 feet bgs in this isolated area during the wet season, but was not seen to that degree elsewhere. Varying levels of redoximorphic mottling was observed in soils throughout much of the study area, at depths between 2 feet and 4 feet, also indicate a history of cyclic wetting and drying associated with seasonal groundwater fluctuations, or transient water flow through the upper subsurface. The sloping site profile likely precludes significant perched water table development within the study area. However, some localized areas may be subject to perched water build-up due to depressed or confined areas of topography and the prevalence of restrictive glacial soils or rock at depth. The site is not proximal to any major natural surface water features.

Conditions observed in test pit explorations are interpreted to be representative of the dry season given the timeframe of explorations in the mid-summer. During the wet season, it is anticipated that groundwater and seepage levels will become elevated from those observed in the summer, and that soil moisture contents will be elevated by prolonged wet weather. The groundwater and soil moisture conditions recorded on our test pit logs are valid only for the dates of exploration.

3.4 Additional Explorations – Sea Pines Road

An additional scope of exploration was requested to document and define subsurface conditions in the area of a proposed sewer improvement near the east margin of the site. The proposed connection for Lot 37 (accessed via Sea Pines Road) plans to extend southwest from the existing 8-inch diameter sewer main current western terminus, through a portion of the easement along the north side of Sea Pines Road, and passing beneath the paved cul-de-sac to connect with the outfall from the project site. The depth to bedrock in the utility improvement area may present a challenge or further expense, and influence the final design alignment and depth.

In-progress plans show the proposed extension alignment will run northeast-southwest approximately 40 feet northwest of the Sea Pines Road centerline. Pipe invert elevation is around 105.5 feet at the existing pipe tie-in (NE end), rising gradually to about 107 feet at the connection to the site outfall (SW end). Where the proposed sewer line crosses underneath the existing cul-de-sac, a minimum of 18 inches of cover will be maintained as required. One or more additional manhole structures may be installed in conjunction with this extension. Base elevations of manhole structures would be in the realm of elevations 105 to 106 feet.

3.4.1 Methods

Subsurface explorations were performed in the vicinity of 315 Sea Pines Road on November 13, 2020. Weather at the time was intermittently rainy. Two (2) test pits (TPs) were machine-excavated in the grassy area north of the cul-de-sac. Two (2) supplemental hand auger borings (HAs) were completed in the area just west of the cul-de-sac during the field visit. An aerial photo site map showing the Sea Pines TP & HA locations (Figure 8) and surveyed topography, subsurface TP and HA logs, and a field photo array (Exhibit B) are attached in Appendix II.

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3.4.2 **Subsurface Conditions**

Test pits were excavated to depths of 6.8 feet bgs (TP-1) and 6.6 feet bgs (TP-2). Organic topsoil and silty/clayey sand were found to overly bedrock at TP-1, and dense glacial till at TP-2. The thickness of cover soils was around 5 feet in each location. The upper soil was generally medium dense and damp to moist or locally wet, containing 20% to 50% fines and exhibiting variable levels of plasticity as interpreted in the field.

Bedrock conditions in TP-1 were observed in the southern (downslope) wall of the pit at a depth of 5.5 feet bgs, and were also present at the central base of the pit. Bedrock was composed of dense, dark gray, medium to fine-grained intact sandstone. Although bedrock conditions were not directly observed in TP-2, it is likely that the dense till material is a thin mantle that is underlain by rock, as seen in numerous other test pits performed within the study area to the west. Shallow seepage was observed between 2.5 feet and 3.5 feet bgs in TP-1, and between 1.2 feet and 2.5 feet in TP2. Seepage appeared to be constrained to the upper soils in the test pits, with moisture content decreasing at depth. Explorations were done in the late fall shoulder season; seepage levels are, therefore, likely elevated from dry season conditions, but not necessarily representative of fully developed wet season conditions.

One hand auger boring (HA-1) was advanced horizontally into the slope cut located just west of the driveway for 315 Sea Pines Road. The boring was advanced through silty sand that transitioned into sandstone weathering rind before hitting refusal at 1.0 feet bgs on very dense, apparently intact, bedrock conditions. Bedrock composition in this location was consistent with conditions observed in other regions of the study area, composed of orange-brown to tan, medium- to fine-grained sandstone.

HA-2 was performed just southwest of the cul-de-sac, south of the proposed sewer alignment in a vegetated area. The boring revealed approximately 0.6 feet of topsoil overlying silty sand containing some clay and gravel with the occasional cobbles, similar to cover soils seen elsewhere. The boring was advanced to an end depth of 4.2 feet bgs where refusal was met, apparently due to a large cobble in the subsurface. Although no groundwater was observed in the boring, and no heavy bands of oxidation coloring were observed, light mottling throughout indicates that the soil likely transmits some amount of water at least intermittently during the wet season.

3.4.3 **Utility Construction and Bedrock Profile**

Following field work, test locations were accurately plotted on a survey map to estimate surface and bedrock elevations. Shallow bedrock was discovered at the toe of the slope in HA-1, around elevation 117 feet, near where the utility will exit eastward from Lot 37. Dense rock conditions were found to be present approximately 1.0 feet into the slope at this area. Whereas, HA-2 found no bedrock through 4 feet depth (roughly elevation 106 feet), suggesting depth to rock can be highly variable along this area at the base of the slope. This could present challenges for excavation to planned utility bedding depth if rock is present at final design location and depth.

At the northeast side of the cul-de-sac, termination of TP-1 was around 5.0 to 7.0 feet bgs on bedrock (elevation 108 feet to 110 feet). At TP-2, bedrock was not present through 110 feet elevation. With an invert elevation in the realm of 106 feet, construction of the sewer outfall line and related structures may contact and require removal of bedrock on the order of a few feet thickness, or less.

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4 Geologic Hazards & Slope Stability

4.1 Review Methodology

The presence and condition of delineated potential Landslide Hazard Areas (LHAs) within the project development area was reviewed as part of this feasibility-level study. As noted in Section 2.4, portions of land within the project site and bordering areas exhibit topography with the combination of height and grade to be defined as *potential* LHAs. The occurrence of defined LHAs is common for hilly areas dominated by bedrock geology in our region, but does not necessarily portend a high or imminent risk of failure. Nor does it trigger blanket avoidance requirements that prohibit construction. Rather, these features are examined on a case-by-case basis to assess the actual hazard presence or potential thereof, and to formulate recommendations for informed development to minimize the risks associated with these natural conditions.

Detailed lot-specific review and exploration for final design recommendations for structures is outside the scope of this study. It is our understanding that lot-specific investigation of subsurface conditions for final design and building permit review will typically be completed individually by the owner at the time of lot development (as is precedent). A lot-by-lot review of existing geohazard features can be completed as needed for the plat approval process under an additional scope of work, if required. A discussion of further work anticipated is included in Section 4.4.

Element Solutions has performed a large-scale feasibility-level assessment of on-site geologic hazards which has included the following components to date:

- Image interpretation and identification of areas of interest for field review (4.1.1)
- Consideration of potential failure mechanisms and contributing geologic conditions (4.1.1)
- Reconnaissance of vegetated/forested slopes to assess for signs of instability (4.2)
- Detailed observation and structural measurement at several steep bedrock outcrops (4.3)
- Graphical analysis of bedrock structures and outcrop stability factors (4.3)
- Determination of actual hazards and recommendations for setback/avoidance (4.4)

4.1.1 Stability Factors and Areas of Potential Hazard

The findings of subsurface explorations and our observation of local exposures indicate that the site is capped by various shallow soil deposits and underlain by folded and tilted sedimentary bedrock of the Chuckanut Formation. We infer that large-scale deep-seated, or global, stability at the site vicinity is controlled and influenced by bedrock structures. Thus, the orientation of rock structures in reference to topography is the primary factor for slope failure modes. Conversely, the stability of shallow soils at a given location is a function of several factors including the character of local deposits, presence of groundwater and potential for runoff inundation, steepness of grade, and stabilizing vegetative cover. As the underlying rock profile limits the depth of a potential failure, the most likely types of failures in cover soils include shallow slumps, translational slides, and saturated mudflows. The most common trigger for shallow instability is oversaturation by groundwater or runoff. Larger circular failures in the site vicinity may be possible where capping glacial soils are thick, or where the underlying bedrock is sufficiently weak/fractured to behave like a soil mass (not observed). Neither condition was found in test pits, although the noted apparent historical landslide area at the northeast corner of the site may have been influenced by a combination of these factors.

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Upon reviewing slope gradient and LiDAR maps, we identified several features for particular focus during reconnaissance. The features occur *within and bordering the proposed development area*, are indicated on the annotated site map (Figure 9, Appendix III), and include:

- Northwest-facing forested slope in the NW quadrant of the site
- Various localized western and central vegetated slopes
- Northwest and west-central steep southeast bedrock faces
- Southeast-facing forested slope within the SW portion of the site

As discussed in Section 2.4.2, we also considered the presence of two specific hazard areas at the margins of the development area (Figures 4 and 5). These features represent known or suspected geologic hazards that may influence the site's final development approach. The coastal southeast slope downhill of the project area is a steep and prominent topographic feature that commonly exceeds 100 feet in height. An apparent historical landslide area is present at the northeast corner of the project area. Reconnaissance and direct observation of these bordering areas was limited or not possible within the scope of this study due to safe access difficulties. Given their location relative to proposed development features, the current review relies upon inferences from site geology and LiDAR image interpretation to set conservative setback standards.

4.2 Slope Review & Observations

During our subsurface exploration program, and following visits for examination of identified areas of interest, an Element Project Geologist and Licensed Engineering Geologist observed conditions of the vegetated slopes among the project area. The purpose of our assessment was to evaluate the present-day stability of the site slopes, and to assess for the presence of indications or features associated with past instability. We traversed the slopes of interest on foot, noting topographic and vegetation patterns and searching for the presence of failure features such as scarps, eroding gulleys, hummocky accumulation zones, etc. Element staff also photo-documented representative slope and bedrock outcrop conditions (Exhibit C). The following subsections address conditions observed by referenced area.

4.2.1 Northwest Slope

This slope is generally planar on the northwest side facing Viewcrest/Fieldston Roads. Elevation increases continuously to the southeast from about 230 feet at the base to about 350 feet maximum at the crest of the slope over a distance of about 250 feet for an average slope ratio of approximately 2:1 (H:V). Statistical analysis of the entire backslope area indicates a mean slope grade of around 50% (~27 degrees). In our experience, this grade is typical for forested, bedrock-controlled slopes in the region.

A predominant majority of the slope area is covered by an established tree canopy, and is vegetated with ferns and other native shrubs. Although many of the trees on the slope were growing straight, some displayed pistol-butt profiles, leaning trunks, and exposed root, indicating that some degree of long-term shallow soil creep is occurring (as is common for steep slopes). The lack of adequate rooting depth may also be contributing to tree orientations, independent of the soil creep phenomenon. Many trees were seen along the edge of the rock cliff face, indicating a stability in the underlying earth material on the plateau of the hill. While some small alders were observed to have fallen from this area, it is likely due to windthrow and a shallow root system, rather than general instability (Photo 1, below).

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Based on vegetation patterns, GIS data analysis, and field observations, the northwest slope appears to be in an overall stable condition lacking signs of large-scale or local instability (aside from typical soil creep). The ground surface is well vegetated, and free of signs of heavy localized erosion or channeling of runoff. Where the ground surface was visible, we did not see indications of slope face retreat, serration or tension cracking, or subsidence that would indicate episodic movement. Some local evidence of historical rock-fall debris was observed near the base of the northwest slope face, but the incidence of fall did not appear to be high, and fallen materials did not extend far from the slope. No ponding, saturation, or seepage was observed above or on the slope during our visits in the summer 2020 season.

The opposite southeast side of the northwestern hill exhibits a steep or cliff-like face with prominent bedrock outcrops. Similar conditions are present to a lesser magnitude along the southeast faces of multiple smaller hills in the central project area. The cliff-formed faces are typically continuous for around 10 to 20 feet maximum and interspersed or bordered with vegetated steep slopes. Small scale rock-fall was observed along the southeastern side of two of the prominent ridge features in the central region of this area, interpreted to be occurring at a low rate of regularity. Detached bedrock blocks were not observed to have traveled far from their points of origin on the outcrops.

Photo 1: Bedrock Cliff Face; Northwest Slope

The cliff area along the northwest hill represents the greatest exposure and highest hazard potential for associated rock-fall (Photo 1, above). At its steepest point, the elevation drops about 37 feet over a horizontal distance of ~25 feet for an average slope ratio approaching 1:1.5 (H:V) along the cliff face. Grades range up to approximately vertical, and are locally overhanging on the variable outcrop faces. We observed these features to be highly influenced by the regularity and orientation of rock structures dictating their stability and character. Section 4.3 below provides a review of bedrock features and structures.

4.2.2 Western and Central Slopes

Select slopes among the middle western and central regions of the site display topography meeting the definition of a critical area slope. These slopes are similar in character to the dominant northwest slope, but occur on a smaller scale interspersed within areas of relatively gentle grades (15% to 30%, or under 15%). Topography appears to be bedrock-controlled, with steeper faces, locally cliff-formed, outcropping on the south or southeastern side of the raised areas. The steeper faces, where grades are greater than 80%, are only continuous for around 10 to 20 feet maximum. Small-scale rock-fall evidence was observed along southeastern side of two of the prominent ridge features in the central area. Similar to the northwestern area, detached blocks and rocks were not observed to have traveled far from their points of origin.

The landforms of interest consist of local rises on the order of about 20 feet maximum expression in relation to surrounding topography that is more gently rolling or sloping. With exception of the

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noted cliff faces, slope gradients are in the range of about 2:1 (H:V) up to 1.5:1 locally. At the top of each local slope area, is plateau or bench topography of low grade. Vegetation is well-developed forest with mature trees and typical undergrowth. During representative reconnaissance of the vicinity, we saw no obvious indications of instability or excess erosion occurring on the steeper grade areas. There were no features identified that would constitute an active geologic hazard.

4.2.3 Southwest Slopes

Slopes flanking the southern project area can be divided into two areas with distinct character. The upland southwest slope begins within the proposed plat lot area and descends with some local breaks, at a predominantly moderate grade, down to a large gentle bench of variable width. The lower coastal southeast slope below the bench descends steeply from crest to shoreline.

The lower coastal slope was identified as a special geologic hazard area recommended for avoidance, with character overviewed in Section 2.4.2. The plat development proposes an "open space" tract along the entirety of the crest of this feature. The proposed lot layout also provides room for substantial setbacks of residences from the lower slope crest (roughly 200+ feet at all lots). We conclude that the proposed layout meets the preferred "avoidance" of the hazard area as well as a reasonable buffer zone. No detailed reconnaissance-level assessment was conducted.

The upland southwest slope generally consists of a series of smaller banks and narrow benches along its upper third (near proposed building areas), followed by more continuous sloping grades downhill. Intermittent slopes on the upper part are roughly 10 to 20 feet high and around 2:1 (H:V), up to 1.5:1 or steeper locally. Benches are on the order of 10 to 20 feet wide with grades under 30%, or below 3:1. The slope and bench topography appear to be controlled or influenced by underlying large bedrock structures, which outcrop locally. Below the second bench (downhill of all proposed building areas), the slope falls at grades of around 2:1 for approximately 50 to 60 feet of elevation until transitioning into the large lower bench of the site (outside of project area).

Topographic contours and LiDAR imagery illustrate that the southwest slope is a generally planar feature; aside from the bedrock-influenced benches breaking the upper third into multiple smaller banks. There are no obvious geomorphic features on the slope suggesting a history of slope failure or channelization of the slope face. There are no apparent head scarps or bowl-shaped features. During reconnaissance, we did not observe any indications of historical or active instability. The slope is well-vegetated with mature forest growth. Trees are generally straight or have minor curvature/tilting attributed to typical soil creep phenomenon.

Aerial photo imagery of the shoreline area was acquired for calendar years 1977, 1994, 2001, 2006, and 2016 to assess for indications of changes or evolution among the southeast slope and coastal area. All images were retrieved from the Department of Ecology Shoreline Photos collection (accessed online). The photo series illustrates that the shoreline and upslope site conditions have not changed appreciably over the preceding 44-year timeframe. Contemporary site conditions appear relatively unchanged from photos taken in years past, and no major clearing or site alterations were observed in the southeast upland area. No obvious indications of mass wasting, such as land scars or loss of vegetation on the slope or shoreline, were observed within the site or surrounding area throughout the period of photo-record. Based on the photo record, we interpret that the shoreline has not undergone visible retreat and that slopes along and above the coastline have remained generally stable over the last 44 years.

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4.3 Bedrock Outcrops & Structures

During reconnaissance, several prominent rock outcrop slopes or cliffs were identified that corresponded to areas of steep to very steep topography indicated by imagery. An Element Project Geologist and Licensed Engineering Geologist returned to the site for detailed observation and direct measurement of the character and structures of the exposed bedrock. We also noted the patterns of rock debris, including extent, size, and relative age, associated with rock cliff areas.

Rock character, intactness, and structural features were examined and documented on the individual outcrop scale (Exhibit C). We measured representative structures with a 360 Azimuth Brunton compass, noting strike and dip of planar features. Rock structures measured included primary bedding, main and secondary jointing patterns, and other planes of weakness if present.

4.3.1 Bedding

Within the project area, ***bedding strikes roughly east-west to northeast-southwest, dipping north and northwest at moderate to steep angles***. According to geologic map resources (e.g. Lapen, 2000), the site lies along the north limb of a broad anticline that traverses the ridge of Chuckanut Mountain, in a northwest-southeast trend, before bending west through the north end of Chuckanut Bay. The hinge of the anticline plunges moderately westward, creating an elongated "V" pattern of major bedding structures and oblique bedding orientations that change by location relative to the hinge. At the site location north of the hinge, bedding is dominantly north- and northwest-dipping. This site-scale pattern can be seen on LiDAR imagery (Figure 4) where resistant beds outcrop or directly influence topography. At the east part of the site, bedding is close to an east-west strike, whereas the west part of the site exhibits northeast-southwest striking topographic features interpreted to be representing or influenced by bedding planes.

It is not clear why the bedding orientations and outcropping patterns are irregular within the site, and outside the scope of this work to further assess. Variations in bedding may be attributed to natural variance in folded rock, since the planar orientation does not range more than about 10 to 20 degrees in each direction from a rough-average ENE-WSW strike. It is also possible that more complex secondary folding is present, and/or that the western part of the site is approaching the fold hinge and reflecting the hinge orientation in part. Also unclear is why the prominent rock faces are isolated and discontinuous in the uphill half of the site, while the rock patterns and outcrop style are relatively consistent along the southern margin and coastal area. It is plausible that the upland area was more heavily affected by the advancement of glacial ice over several ice age episodes. While glacial deposits are relatively thin, the effect of rock erosion during glacial advance may have been significant enough to alter the upland landscape.

Generally speaking, the major bedding orientation (dipping northwest, into hillsides) is favorable for site slope stability. We examined this relationship and variations on the outcrop scale. Bedding on the large northwest cliff face ranged in strike from 220 to 255 degrees (360 Azimuth). Dip of bedding at the northwest outcrop was between 40 and 60 degrees (Figure 10a). Bedding on the smaller west-central outcrops was either broadly similar (west location) or progressively east-west striking (east location). Both outcrops exhibited bedding that was relatively steeper than at the northwest cliff; measured dips ranged from 55 to 65 degrees (Figure 10b). Converse to the bedding, outcrop faces were oriented NNE-SSW or NE-SW and moderately steep to steep overall facing to the southeast. ***At all locations, bedding is oriented nearly opposite to the exposed face***.

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4.3.2 Joint Patterns

In the folded Chuckanut Formation, it is common to observe one or more brittle joint orientations that occur in a discontinuous, but regular interval on the one-foot to several-meter scale. These planes of weakness are also common enough to influence rock slope stability. In our experience, the primary joint plane is often roughly perpendicular to the bedding orientation, occurring as a result of folding and/or compression of the unit during deformation. One or more secondary joint orientations may be oblique or perpendicular to the first joint set and/or bedding. These are often attributed as bedding expansion joints and, therefore, form weaknesses near orthogonal to the bedding itself but are confined within bedding layers. The result of one or multiple joint patterns on slope stability can range from relatively nil to major depending on joint orientations versus each other and the exposure plane.

In the outcrops, the ***main joint pattern was observed to be steeply to moderately dipping west or southwest and striking NNW-SSE or NW-SE. The dominant orientation is normal or oblique to the exposure faces, and is close to orthogonal to average bedding***. At the northwest cliff face (Stereonet Figure 10a), the main joints were near-vertical and one companion joint was measured (same strike, dipping opposite direction to NE). At the central outcrops, the main joint planes were typically steeply to moderately dipping to the SW (Stereonet Figure 10b). Joint structures are shown as dotted planes with bedding as solid lines in the attached Stereonet diagrams.

Multiple ***secondary joint or fracture orientations were also measured*** at each outcrop area. We note that these features tended to be smaller, discontinuous planes or open-face fractures that are poorly defined, and thus they do not necessarily represent a major discontinuity structure. However, they can have an influence on outcrop-scale processes such as rock fall hazard. Open planes were observed dipping steeply south or SE in a similar or oblique orientation to the outcrop (possibly influencing the outcrop orientation). These were characterized as rock fall breakage surfaces (see discussion below). We also observed a sub-horizontal joint plane along the northwest cliff face that was not observed elsewhere and may be relatively rare or inconsequential.

Finally, we observed for obvious indications of joints intersecting in unfavorable orientations contributing to rock falls or slides. Excluding the subparallel-to-face joints, ***we did not observe wedge or triangular joint patterns in the outcrops*** that could be associated with a non-planar failure system. This is consistent with our graphical interpretation of joint patterns and orientations relating to wedge failure (discussed in 4.3.3).

4.3.3 Rock Face Stability

Strength of a rock mass is controlled and limited by internal structures that are planes of inherent weakness (bedding/foliation) or fractures (joints, veins, faults), rather than rock strength itself. Inherent planes are penetrative, while fractures tend to be discontinuous but regular in occurrence. Orientation of structures with respect to the slope face influences the potential for various styles of rock slope failures. Major failure types include planar sliding (along a continuous bedding or fracture plane), wedge failure (intersection of two planes forms sliding angle with respect to outcrop), and raveling or toppling (intermittent mass wasting parallel to face, style depends on rock type). Each type of failure is discussed below in terms of its interpreted potential at outcrops on site. Interpretations are adopted from Wyllie & Mah (Rock Slope Engineering, 4th Edition, 2005), based on prior work of Hoek and Bray (1981) for rock slope stability. Stereonet plots (Figures 10a & 10b) were used for graphical analysis and interpretation of failure modes.

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PLANE FAILURE:
Planar failures can theoretically occur where a sliding surface emerges on a steeper exposed face. The sliding surface must be dipping greater than the rock's friction angle (commonly between 30 to 40 degrees for granular sedimentary rock). The reference text notes that pure planar failures are rare, as they demand several unfavorable boundary conditions to be met in addition to the correct plane orientation. Planar failures are also limited to planes within about 20 degrees strike of the exposure.

Outcrops and slopes at the site are not at risk of planar failure from the bedding or primary joints. Bedding dips in the opposite direction of the cliff exposure slopes, and the main joint planes are nearly orthogonal to the slope face. Secondary joint and breakage faces are considered small and discontinuous, and not inherently at risk for sliding failure.

The northwest slope face is oriented similarly to bedding. We surmise that the slope form is influenced by rock bedding. However, the condition does not represent a dip-slope hazard. The topographic slope incline is less than the bedding orientations observed, so that bedding submerges into the ground as opposed to emerging from the slope at a lesser angle.

WEDGE FAILURE:
A wedge failure mode can be created along the intersection of two planes of weakness when the intersection line of the planes satisfies criteria for sliding relative to the slope face, even if the planes themselves would not. Again, the intersection must slope greater than the friction angle of the rock discontinuity and daylight on the slope in an orientation close enough to the slope dip.

We examined potential wedge failure modes resulting from joint-to-bedding and joint-to-joint interactions at the site. The ***main intersection of bedding and joints plots in the northwest quadrant of the Stereonets, and plunges moderately to steeply northwest*** (Figures 10a & 10b), thus into the steep outcrops. Other intersections with bedding and shallow joint planes are all at low angles which do not pose a risk of sliding. While this avoids direct wedge failure, we note that the steep intersections could contribute to small-scale rock fall in the opposite direction when paired with other factors including cliff exposure.

TOPPLING/RAVELING:
Failure by toppling or raveling does not require a sliding scenario, but can occur under a variety of circumstances which vary in severity and regularity by rock type. A key factor for this type of failure mechanism is the presence of a steep, sub-vertical, or overhanging slope face, along with steep bedding and/or jointing planes. Shallow secondary planes which disrupt the main planes can further deteriorate the rock mass.

We infer progressive raveling and/or small-scale wasting of the rock face is a common and unavoidable occurrence at the outcrop locations within the site. The major bedding planes have been dissected by steep and shallow jointing on the foot- to meter-scale, resulting in exposed rock susceptible to localized raveling over time despite the favorable bedding orientation. However, the presence of the natural cliff exposures indicates the rock mass at these locations is relatively stable and subject to a slow process of raveling, presumably since the last glacial episode.

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4.3.4 Rock Fall Characteristics

Existing rock debris observed on the ground surface in the downslope vicinities of the several exposures is broadly consistent with our interpretation of raveling and small-scale rock breakage as the main mechanism of rock wasting. We have relied on the empirical patterns of prior rock fall observed in the field to inform their occurrence, apparent regularity, and overall magnitude.

Some evidence of incidental toppling was observed near larger rock faces in the northern and central regions of the project area. Fallen blocks were generally observed to be of an elongated shape, and the majority were measured to be from about 1 foot to 3 feet in size along the a-axis. Blocks were observed to be situated around 10 feet to 15 feet maximum from their perceived points of origin. Some larger blocks, around 5 feet to 7 feet along the a-axis, were also observed to have become detached and traveled short distances. The larger blocks were also of an elongated shape, and were only observed to have traveled about 1 to 8 feet from where they had fallen. The non-spherical shape of the blocks is interpreted to reduce the distance of potential translation or runout, along with the presence of thick forest vegetation hindering runout. None of the more recent blocks observed were noted to have fallen more than about 20 feet from the outcrop of origination.

A few relatively medium to large sized boulders were observed in the valley area downhill of the largest outcrop, below the northwest cliff face. These materials were old enough to be partially or mostly buried and covered in moss growth. Their origin cannot be directly confirmed as outcrop rock fall, as they may be an earlier byproduct of historical erosion and/or glacial depostional processes. Even presuming a rock fall origin, the boulders appear to be of significant age indicating a very rare occurrence potential in the time scale of the project.

4.4 Geohazard Review Findings & Recommendations

This study has involved field reconnaissance and graphical analysis to review slope stability factors and evidence of instability considering both cover soil deposits and underlying bedrock. Based on the work completed to date, we have reached the following interpretations and conclusions on project site slope stability (4.4.1). These conclusions form the basis of preliminary recommendations for building setbacks, mitigations, or development limitations with respect to specific site features (4.4.2). We also address the need for further lot-specific reviews for design and permitting of individual SFR developments. This section focuses on setbacks for building features (structures, roads, etc.). For discussion of stormwater management features placement with respect to slopes of concern, see Sections 5.1.3 and 5.12.1.

4.4.1 Conclusions on Slope Stability for Development

In our opinion, the sloping parts of the site within and in proximity to the proposed development areas (excepting localized steep cliff faces) display characteristics indicating stable conditions are broadly present. Excluding the special hazard areas discussed in Section 2.4.2, recommended to be avoided, we did not encounter evidence of active or historical slope failures, nor areas of excessive erosion. Forest vegetation throughout the site is well established. The combination of grades and subsurface conditions is conducive to maintaining long-term stability of the site with a relatively low risk of instability. The presence, character, and orientation of bedrock underlying the site is also found to be favorable for global stability of the site. Thus, the variable and locally moderate to steep topography intermittent throughout the site should not preclude its development, assuming a proper design and construction strategy is employed.

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Proposed roads appear to be aligned in a manner that avoids excessive cuts or fills on sloping areas, taking advantage of natural benches or valleys in topography. Standard cut-and-fill practices and roadside bank constructions are anticipated to be feasible, as addressed below. Small retaining structures can be employed as needed where space is constrained. Roads and driveway extensions have preferentially avoided areas of steeper grades, where possible. The roads do not pass in close proximity to the delineated special hazard areas. Major utility services will be predominantly constructed along the road corridors and protected from slope processes.

The anticipated building areas on individual lots will deal with a variety of terrain situations. In our experience, the combination of topographic challenges and subsurface conditions are not uncommon for home site development in the Cascade foothills within and surrounding the Bellingham area. The blanket code definition of portions of the project site as geologically hazardous areas based on slope grades should not prevent appropriate use on the lots involved. It is expected that individual lot home designs will incorporate foundations that are best fit to the topography. Multi-tier footing systems, foundation retaining walls, and daylight basement features are commonly used to construct homes on topography similar that present on the project site. The soil and bedrock conditions are considered broadly well suited for these approaches to be adopted on a per lot basis during future design and construction.

4.4.2 Preliminary Building Setback & Avoidance Recommendations

Based on the feasibility-scale review completed to date, we recommend the following guidelines for plat planning and individual lot building placement with respect to geologic hazard features. Note that some locations are referenced below to the most current proposed plat layout.

1) Generally speaking, unless otherwise addressed below, ***areas within the development zone exceeding the 30% (erosion hazard) and 40% (potential landslide hazard) thresholds per code do not require avoidance or setback criteria***. Rather, we recommend development of the areas adhere to best management practices for slope-side design and construction typical for this area. For instance, homes should be carefully sited and designed where steep grades are present to ensure long-term stability of slopes and structures. Local adjustments may be necessary to avoid small-scale features not fully evaluated in the scale of the current work.

   Foundations on or near slopes will require embedment and suitable placement on stable subgrades to avoid unacceptable risk. Cut-and-fill leveling of building sites on slopes is not recommended. The use of heightened stem walls, stepped or tiered foundations, and retaining wall features is typically preferred to bank modifications and fill pad construction. In addition, site preparations and restoration measures (erosion control, planting practices, stormwater drainage controls, etc.) must adhere to critical area protection measures as overviewed in Section 5.12.

2) Local rock cliff features are recommended to be avoided by incorporating an appropriate setback to building foundations. The setback can be defined by distance from the slope crest above the feature, or from the relative foundation placement depth and location with respect to the outcrop exposure if the approximate building location and design style are known. For the current purposes, we preliminarily recommend setbacks based on horizontal distance from a slope crest irrespective of design. ***The recommended setbacks should be reviewed and adjusted as necessary during individual lot design***.

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We recommend preliminary minimum horizontal building setbacks from the northwest hill southeast cliff face of 30 feet for Lot 8 and 20 feet for Lot 9, the proposed lots located on the narrow ridge. A preliminary 15-foot minimum foundation setback is also recommended for Lot 14, which is located on the uphill side of the west-central steep rock outcrop. The last notable outcrop, generally located at Lot 20, is smallest in stature and may be partially abated by building pad earthwork. Where steep exposed rock remains below the building area, a minimum 10-foot foundation setback from exposure is recommended. These preliminary ***setbacks equate to an approximate 1:1 distance versus height of the underlying steep outcrops***. In our opinion, this is a conservative approach that will provide ample building protection from future potential of instability and periodic rock face loss over the long term.

3) Due to the potential for incidental rock fall from the several outcrop faces, ***we recommend ample avoidance or protective measures be incorporated for areas immediately downslope of cliff exposures***. For the current proposed layout, home sites that may be directly affected by rock fall include Lots 21 and 22. For full avoidance without need for other mitigative measures, a ***minimum separation of 15 feet from the underside (toe) of the exposed rock face is recommended at these locations***. If home construction is elected or required to be closer to the rock face, use of a separate catchment structure (such as a landscape wall with some free height) or incorporation of a heightened reinforced foundation wall is advised. We recommend the conditions be reviewed in detail on an individual lot basis, where necessary during lot-specific design, and that final recommendations for rock fall avoidance or mitigation be issued at that time based on the proposed building layout.

   Road and driveway areas may also be subjected to rock fall where in close proximity to the outcrop faces. Areas of potential concern include the primary access "West Road" traversing the valley area below the large northwest outcrop, the attached small driveway access to Lots 16/17/19/20, and the cul-de-sac of the "East Road" below the central small outcrop. However, with the interpreted rare regularity and low potential for significant runout of rock-fall debris, extensive mitigations do not appear necessary. We advise considering incorporation of a topographic swale or low catchment wall on the uphill side of the "West Road" and the "East Road" cul-de-sac to safeguard from incidental rock-fall reaching the roadway and intersecting driveways. If the road alignment is adjusted to be farther from the cliff feature, these measures can be avoided. Alternatively, as-needed rock fall cleanup and repair could be done in exchange for up-front mitigations where construction is costly or limited.

4) The coastal southeast slope and its upland vicinity is recommended to be fully avoided by development. For general planning purposes, we recommend applying a non-development building buffer equivalent to the slope height. Total height varies locally from about 100 feet minimum to around 150 feet maximum. The current proposed layout allows for over 150 feet separation to building zones at all areas, consistent with this guideline.

5) The northeast corner area, interpreted as a possible historical landslide area based on geomorphic features, is recommended to be avoided. Per City of Bellingham code, the standard minimum setback from active or historical LHA features to developments is 50 feet. The current plat layout allows for ample setback to upslope areas. This setback can be investigated further on a per-lot basis during lot design, and may be eligible for reduction upon demonstrating adequate factor-of-safety is achieved at a lesser distance.

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4.4.3 Need for Lot-Specific Reviews

The site-wide geohazard review completed to date represents an overview of site features with specific attention paid to potential hazards identified along the boundaries of or intermittently within the large hilly property. It is not intended to serve as a detailed examination of the conditions on individual lots to advise on lot designs. Based on our experience, it is most appropriate to conduct detailed evaluation of topographic and subsurface conditions on individual lots in the future just prior to or during their design and development when proposed features and final layouts can be taken into account.

We recommend that all lots containing or bordering potential LHAs (as code-defined, grades over 40% and relief over 10 feet) be required to conduct lot-specific final critical area reviews at the time of building permitting. At minimum, a reconnaissance-level assessment and review of proposed building plans should be completed. We recommend site evaluations include subsurface exploration to assess foundation conditions and prescribe foundation design/construction recommendations for any building areas on or directly adjacent to slopes over 40% grade. Future studies should be responsible for either confirming the findings and recommendations of this report, including setbacks if applicable, or offering new or revised recommendations based on detailed assessment of a lot.

To some degree, further lot-specific review and critical area documentation can be completed supplementally to this report. Some portions of the site can also be addressed in kind (such as lots at the base of the northwest hill, and lots lining the top of the southern slope). If further detailed lot review is required for plat approval or requested by the client, Element Solutions will be pleased to provide the additional assessment on a per-lot basis.

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5 Conclusions and Recommendations

5.1 Project Feasibility Discussion

Based on the findings of our site-wide subsurface investigation, geologic hazard assessment, and the interpretations presented herein, it is our opinion that the proposed plat development is feasible as generally proposed. We recommend following the guidelines and recommendations below for plat design and construction. We anticipate conventional design and construction practices will be suitable for this project, assuming a typical level of risk is acceptable.

This study was conducted as a feasibility-level evaluation for the plat, and is not intended to present detailed information for individual lot constructions. In this section, we provide preliminary commentary and general design guidelines for development. On the per-lot scale, the information may need to be expanded upon or modified to address lot-specific conditions. Detailed work done at a later date by Element Solutions or another qualified geotechnical consultant may supersede the broadly based recommendations of this report.

5.1.1 Foundation Feasibility Commentary

For a shallow foundation to be feasible, adverse levels of settlement must be avoided. This requires that either the ground conditions below the structure are suitable for supporting anticipated loads without inducing excessive settlement, or that site preparations and/or design factors are incorporated to minimize inherent settlement risk to an acceptable degree. Settlement can be a result of shallow factors (organic or soft/loose subgrade, uncontrolled or improperly compacted fill, erosion of support, etc.), deeper factors such as soft-soil consolidation, or a combination of both. Foundation settlement can also be associated with sloping grades and insufficient embedment or bearing support.

Native soils at the project site are generally well-suited for residential building foundations and pavement development. The soils are not excessively moisture-sensitive, nor are they of excessively soft consistency or loose density. Shallow deposits are locally variable, however. Shallow saturation in the winter season (caused by underlying restrictive conditions) can also pose a risk for moisture-sensitive subgrade deterioration from freeze-thaw effects. These factors can be mitigated to a reasonable level by careful site preparation to minimize variability and ensure proper subgrades are established. In addition to the prescribed site preparations below, some localized over-excavation of problematic subgrades may be needed during site preparations and home foundation constructions.

With the exception of surficial topsoils and rare historical grade fills at shallow depths, no unsuitable or highly compressible soils were encountered through maximum depth explored. Additionally, the site subsurface is not susceptible to excessive settlement during a seismic event. There are no concerns for loss of building support associated with deeper conditions given the underlying dense to very dense glacial drift/till and bedrock profile throughout the site.

Based on the findings of field explorations and analysis of the site conditions, it is our opinion that shallow footing foundation systems are feasible for the proposed project. In Section 5.3, we provide preliminary foundation design and construction recommendations tailored to the subsurface conditions documented in the site-wide test pit survey.

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5.1.2 Road & Utility Construction Feasibility

The primary challenge for road and driveway construction within the development is the prevalence of variable surface grades, even along the optimal alignments proposed with the plat layout. We expect cut-and-fill grading will commonly be necessary along the length of roadways. Most grade adjustments will be on the order of a few feet. Maximum fill thickness is anticipated to be in the range of 5 to 7 feet locally. Some road areas will also be dealing with off-camber, or cross-sloping, topography. It is recommended to build road sections in full cuts or fills, and to avoid partial cut-and-fill transitions where feasible. Where transitional areas are unavoidable, we recommend additional site preparations to properly bench subgrades for fill placement along with diligence in compaction of base materials below and along the side banks of the road to minimize the risk of future road settlement due to partial fills. Utilities constructed below partially filled roadway areas should preferably be placed at depth within underlying native soils to ensure that the integrity and performance of the line is not adversely affected.

Depending on depth of road cuts and utility installs planned, some areas may encounter bedrock before target depth of excavations. Sandstone bedrock was commonly encountered by about 4 to 5 feet depth at most test pit explorations along the entry corridor and "West Road" alignment in the north- and west-central regions of the site. Locally, bedrock was present within about 2 to 3 feet depth along the "East Road" alignment and cul-de-sac. At TP-4 in the east-central area, bedrock was found directly below topsoil. Refer to Figure 7 for illustration of depth to bedrock by test pit location. In our experience, rock excavation for utility installs and local subgrade leveling in Chuckanut Formation bedrock is relatively difficult where intact sandstone is present, and moderately difficult where rock is composed of fractured sandstone or siltstone. Conventional equipment can be used with rock breaking attachments, but the process can be time-consuming. It is recommended that subsurface data be carefully reviewed for design and construction planning so that major conflicts with rock depths can be avoided. Additional targeted explorations should be done if needed to better define depth to bedrock at certain areas for utility construction.

5.1.3 Stormwater Infiltration Design Feasibility

The project will be required to manage stormwater from new impervious surfaces in accordance with the Department of Ecology Stormwater Management Manual for Western Washington and its local municipal application. In this study, the general feasibility of on-site stormwater infiltration was evaluated in accordance with current City of Bellingham pre-permit review standards. Alternatives such as on-site dispersion and tightline outfalls were also considered.

Due to topographical and surrounding development constraints, we understand primary stormwater management for the project's interior infrastructure and building lots will generally need to be either handled within the property, or directed via tightline down the coastal slope to the southeast shoreline for release. Stormwater management of the site in majority will most likely entail collection/detention of runoff from pavements and structures, then tightline conveyance to suitable upland dispersion areas and/or by a primary outfall pipe down to the coastline. A combination of factors such as limited lot sizes, variably sloping topography, and proximity to other homes and roads will preclude use of dispersion on most individual lots. Northerly areas of the site may drain separately out to Viewcrest Road.

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While there are some localized opportunities that could be pursued for small-scale infiltration on lots, as discussed below, the predominant majority of the site is not conducive to infiltration due to shallow restrictive soil/rock conditions, potential for perched seasonal groundwater, steep grades with potential for saturation-induced instability, or a combination of limiting factors. Local infiltration, where viable, is best suited for individual lot stormwater management at select areas to be addressed with future design and construction of home sites. Aside from the localized infiltration usage, most lots are recommended to have runoff captured and routed for dispersion or off-site disposal.

Potential Residential Lot Infiltration Areas

The northwestern and north-central portion of the property in the vicinity of Viewcrest Road was interpreted from exploration data to have the best potential for per-lot infiltration. This area generally consists of approximately 1.5 to 3.0 feet of cover soil and 1.5 to 3.0 feet of glacial outwash overlying glacial drift or till. The outwash material consists of sand and gravel with a generally low fines content and relatively high natural transmissivity. Analysis of infiltration capacity for the outwash-type soils found locally is presented in Section 5.7.

The project is within the City of Bellingham jurisdiction, which stipulates that at least 3.0 feet of permeable soils and at least 1.0 feet of separation must be available for residential downspout infiltration systems to be feasible. Typical options include linear trenches or drywells. The soil profiles observed in TP-13 through TP-17 (Lots 1 to 7 area) all appear to meet or exceed these criteria, where explored. The northwest and north-central areas also generally grade down to the north, separate from the majority site topography. Therefore, stormwater infiltrated locally on these lots will not place a hydrologic load on sensitive slope areas.

Pursuant to local stormwater regulations, which dictate residential lot infiltration systems be used *where feasible*, we recommend infiltration systems be considered on these northerly lots/areas in the future during final lot design. The actual application will depend on other factors, including grading, space, and conditions at areas open for stormwater use on each lot. We recommend a contingency plan of off-site tightline disposal in the event that infiltration is found to be non-viable upon further review on a per-lot basis. A public stormwater utility is mapped along the south side of Viewcrest Road directly in front of lots in the referenced area that may be an option for off-site stormwater disposal.

5.2 Seismic Design and Liquefaction Potential

This section addresses site-modified seismic design parameters based on regional-scale mapping of Site Class and the subsurface conditions encountered in our investigation. Additionally, we address site-specific liquefaction susceptibility.

5.2.1 Seismic Design Coefficients

For structural design purposes, our assessment of site geology may be considered Site Class C, representing a dense soil and bedrock profile. For design code standards per IBC 2018, we have determined utilizing web-based design tools that the following seismic parameters (Table 3) are appropriate for design of the proposed residences. Peak Ground Acceleration values were generated based on a combination of ASCE 7-16 and IBC 2018 guidelines.

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Table 3: Seismic Design Parameters

Coefficient	Description	Value
S_S	Mapped Spectral Acceleration (0.2 second period)	1.018
S_1	Mapped Spectral Acceleration (1.0 second period)	0.358
S_{MS}	Site-modified Spectral Acceleration (0.2 second period)	1.222
S_{M1}	Site-modified Spectral Acceleration (1.0 second period)	0.537
S_{DS}	Design Value (0.2 second SA)	0.815
S_{D1}	Design Value (1.0 second SA)	0.358
PGA	MCE_G Peak Ground Acceleration	0.435 [g]
F_{PGA}	Site Amplification Factor at PGA	1.2
PGA_M	Site Modified Peak Ground Acceleration	0.522 [g]

5.2.2 Liquefaction Susceptibility

Soil liquefaction is a result of loss in effective shear strength under the influence of elevated pore water pressure development during a seismic event. For soils with lower internal shear strength, earth shaking during an earthquake may cause pore water pressures to exceed the strength of the soil and "liquefy" portions of the profile. In general, saturated, loose to medium dense and cohesionless granular soils are most prone to liquefaction. Whereas high-fines cohesive and plastic soils and dense/hard soils or bedrock are not considered liquefiable. Liquefaction can induce total and differential ground settlement, surface disruptions, and lateral spreading where there is a lack of buttress or lateral support (such as near a slope or water body). Liquefaction and seismic shaking can also instigate soil slope failures where global stability of a slope is limited by shear strength. The effects of liquefaction are difficult to predict and can vary locally as evidenced by past events.

The *Liquefaction Susceptibility Map of Whatcom County, Washington* (Palmer et al., 2004) indicates the site vicinity has a negligible potential for liquefaction to occur due to the underlying bedrock geology. The mapping is based on generalizations of subsurface conditions associated with regional-scale geologic deposits, and should be considered on the site scale for potential variations based on exploration data. Our on-site findings have confirmed the map designation of no discernable liquefaction hazard at the site.

5.3 Foundation Design and Construction

For home foundation site preparations, we recommend first removing all topsoil and organic materials, uncontrolled fills or disturbed soils if present, and soft or loose cover soils down to native subgrade of medium dense/stiff or better consistency. Local over-excavation may be required to address problematic areas and variations in the shallow deposits. Recompact granular subgrades to mitigate excavation disturbance and promote a uniform density. Fine-grained subgrades should be protected from excessive disturbance and exposure limited during inclement weather conditions before foundations are installed.

Foundation excavation depths to reach competent subgrade are expected to be typical for shallow construction where building on gentle grades. Where building on grades of 3:1 (H:V) or higher, a minimum embedment of 2.0 feet is recommended for lateral stability and erosion protection. Foundation areas proposed on grades of 40% or greater are recommended to undergo site-specific review and be designed appropriately for slope-side construction. It is presumed that critical area slope evaluations will be required on a case-by-case basis for areas of steep grades.

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We recommend all foundations on sloping topography be constructed directly on native cut subgrades by use of stepped footings or tiered footing levels. This will avoid the risk of differential settlement between foundations supported on native subgrade versus those on leveling fills.

5.3.1 Bearing Capacity

Assuming home site foundation areas are prepared as recommended above, a prescriptive or general ***allowable vertical bearing capacity of 2,000 pounds per square foot (psf) is recommended***. This capacity takes into account the range of native soils present on site, and incorporates a factor of safety of at least 3. Values assume placement directly on medium dense/stiff or better undisturbed native subgrade. The allowable bearing capacity can be increased up to 1/3 to account for short-term transient loading such as associated with seismic or wind loads.
A greater allowable bearing capacity can be utilized where foundations will be placed directly on dense/hard glacial till or bedrock subgrades. In these cases, an ***allowable vertical bearing capacity of up to 3,000 pounds per square foot (psf)*** can be employed. Where increased bearing loads are planned to be used, we recommend that subgrade conditions be verified directly by site-specific evaluation as well as during construction by a geotechnical professional.

Foundations shall be sized sufficiently to meet the maximum allowable bearing load requirements, or to meet minimum size requirements per IBC requirements governing at the time of construction, whichever is larger.

Expected settlements will be largely elastic and well within structural tolerances for the proposed home structures, provided footing bearing surfaces are carefully prepared and not disturbed. Settlements should not exceed 1-inch total, nor ½-inch differential, over 50 lineal feet, within code-defined limits.

5.3.2 Lateral Resistance

Sliding resistance contribution to lateral load resistance applies to foundations placed in contact with the supporting subgrade. For application to either placement on native soils or structural fills, as conditions dictate, a coefficient of sliding friction of 0.30 is recommended for broad use. This value is function of the internal friction of the subgrade soil, and includes a factor-of-safety of at least 1.5. For well-compacted imported granular structural fills placed as foundation base fill, and for foundations placed directly on sandstone bedrock, the coefficient can be increased to 0.50.

Lateral earth pressures imparted and passive lateral resistance provided by foundation backfill are addressed in Section 5.4 Retaining Wall Foundations. The frictional forces can also be applied to restraining scenarios.

5.3.3 Foundation Drainage

The site commonly exhibits conditions with potential for shallow seasonal soil saturation and/or perched transient groundwater. Lots on lower portions of the site may be susceptible to subsurface drainage from the upland vicinity. We highly recommend use of perimeter foundation drains to promote long-term dry foundation conditions. In addition to perimeter foundation drainage, we recommend exterior ground surfaces and pavements be graded to slope away from structures. Building ancillary features should avoid those that could allow water to collect and pond against the outside of the structure. Exterior pavements and flatworks near the structure should incorporate local surface drains to control runoff.

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For greatest effectiveness, footing drains should be placed even with the base of the footing along the exterior of structures. A continuous, 4-inch minimum diameter, perforated pipe that is sloped for gravity-assisted drainage and wrapped in filtration fabric or a filter sock is recommended. The area around the pipe and extending against the adjacent foundation wall should be backfilled with drain rock and separated from adjacent soils by use of soil separation fabric. Unless otherwise specified by design, the upper 1.0 foot of subsurface should be capped by low permeability fill material or pavement to minimize vertical water transmission from the building exterior to the foundation. Connect footing drains via tight-line to a catch basin or discharge facility separately from roof drains and other exterior surface drains to avoid backwards transmission or flooding of the foundation drain system by stormwater sources.

5.4 Retaining Wall Foundations

Retaining wall foundations may be used with some residences to permit construction directly against slope cuts or for daylight basements on sloping grades. In these cases, cast-in-place concrete walls of about 1-story maximum height are expected. This section provides preliminary guidelines and recommendations for structural retaining wall design and construction. Since walls will typically be employed in areas with steep slopes, we recommend lot-specific critical area reviews to confirm or modify the input as appropriate. At minimum, we recommend that Element Solutions be contacted to review proposed design plans and consult on specific applications in the absence of additional investigation.

5.4.1 Lateral Earth Pressures

Wall features in lateral contact with soils are subject to earth pressures and resistances from native soils (cut locations), or as a result of backfill materials placed against them (fill conditions). Recommended static lateral earth pressures (active and at-rest) are summarized in Table 4 (provided as equivalent fluid weight, units psf/foot or pcf). For the seismic design case (^), experience has shown that retaining wall structures perform very well based on designs employing the at-rest earth pressure loading pressures. The provided values assume fully drained conditions and increase linearly with depth. *Undrained design situations must also account for hydrostatic pressure with correspondingly increased values; contact Element Solutions for consultation on design using undrained conditions if required for the project*.

Table 4: Lateral Earth Pressures by Soil Type

Soil	Condition	Soil Unit Weight (PCF)	Active (EFW)	At-Rest^ (EFW)	Passive Lateral Resistance (EFW)
Native Soil (SM – ML) (Silty Sand-Sandy Silt)	Retained	115 - 125	40	60	375* (static) 300* (seismic)
Structural Fill (GP)	Backfill	125 – 135	30	50	600* (static) 500* (seismic)

Values in Table 4 do not include additional pressures imparted from sloping backfills, vehicle loads, temporary stockpiles, or loads from nearby structures. Wall designs must account for adjacent surcharge loads in addition to the model lateral earth pressures. Structural Fill values will typically apply where walls are used to build up from existing grades. The exception is for walls constructed closely to and in part against native soil cuts. In that case, where backfill width is less than wall height, we recommend using the Table 4 earth pressure values corresponding to native soils.

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The passive lateral resistance values for soils in Table 4 are unfactored values*. Appropriate factors of safety should be applied when using passive soil resistance to reduce the parameter to the acceptable design value. We recommend safety factors of 3 and 2 be applied under static and seismic conditions, respectively. For backfills providing passive restraint and extending at least 3.0 times the wall foundation depth horizontally from the foundation, values for compacted structural fill can be used. For lesser supporting widths of structural fill, and for foundations placed "neat" against undisturbed and competent native soils, the corresponding native soil parameters should be applied for passive resistance. All passive restraint values assume a horizontal surface for the supporting soil, and sloping surfaces must be evaluated on a case-specific basis.

5.4.2 Wall Construction Recommendations

A dedicated wall drain system is necessary to promote backfill drainage and minimize hydrostatic pressures behind walls. All walls are recommended to incorporate foundation drains as specified in 5.3.3 Foundation Drainage. In addition, backfill for the first 12 inches minimum behind walls is recommended to consist of fully free-draining material, such as Gravel Backfill for Drains (WSDOT SS 9-03.12(4)), or approved equivalent. We recommend placing filter fabric between the drainage corridor and backfills or retained soils to limit fine material from entering the free-draining zone.

Sealing of home foundation retaining walls with waterproofing treatment is advisable if low levels of potential leakage over time is unacceptable; without treatment, some through-wall transmission during heavy flows should be expected.

We recommend relatively free-draining gravel backfill be utilized within 5 feet of retaining walls. Free-draining materials have a typical maximum of around 3% fines content (depending on material type), and thus standard structural fill may not be suitable. Retaining wall backfill should comply with WSDOT SS 9-03.12(2) Gravel Backfill for Walls, or approved equivalent.

Backfill placed near walls (within about 5 feet) should be compacted with appropriate small equipment to avoid excess compaction leading to potentially elevated earth pressures. Place and compact fills in approximately 6-inch lifts while working progressively further from the back of the wall. Backfill should be delayed until the wall concrete has cured to acceptable strength.

5.5 Slab-On-Grade Floors

A slab-on-grade floor may be used for portions of the home structures. Loading is anticipated to be light residential use; no heavily trafficked or loaded areas are expected. Any slabs that will be subject to high loads or heavy vehicle traffic are recommended to be designed as rigid pavement sections with adequate slab thickness, reinforcement, and base materials for the expected use.

5.5.1 Slab Preparation and Construction

For slab-on-grade areas preparation, we recommend all organic soils and unsuitably loose or soft soils be removed. Granular subgrades should be recompacted after stripping to a uniformly medium dense or better condition. Fine-grained subgrades should be verified as suitably stiff and unyielding. We recommend a proof roll be conducted on slab subgrades, if weather conditions and access permits, prior to capping with structural fill. Any areas identified by proof roll to be loose, soft, or pumping are recommended for over-excavation and backfill with structural fill.

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For the encountered site conditions, we recommend installing a base pad of at least 6 inches minimum thickness below floor slabs. This will promote under-slab drainage and provide stabilization over shallow moisture-sensitive subgrades. Slab base fill is considered structural fill, and should comply with the recommendations below for material type and installation. A properly compacted angular crushed-rock capillary break using structural-quality material (Section 4.4.2) can account for the recommended base section.

Assuming diligent subgrade preparations and recommended base pad installation, we recommend slab design use an allowable Subgrade Modulus (k) of up to 125 pci for design of light-load interior floor slabs.

5.5.2 Slab Drainage and Moisture Control

All interior slab-on-grade floors are recommended to be underlain by a capillary break section composed of appropriate free-draining material. For this purpose, we recommend a 6-inch minimum section of uniformly-graded, low-fines content, angular, clear crushed rock be placed and compacted to a dense and unyielding condition. Capillary break material is recommended to contain at maximum 3 percent fines (amount passing U.S. #200 Sieve) and be composed of 3/4-inch to 1.0-inch clear crushed rock material with nominal content passing the U.S. #4 Sieve. Where composed of approved structural-quality material (as recommended), it can account for the slab base pad.

A vapor barrier is also recommended below interior floor slabs. To inhibit moisture transmission through the slab where floor coverings can be impacted by moisture, we recommend placing a 10-mil or thicker polyethylene membrane below the slab. The barrier should be placed to overlap between sheets and properly sealed at the adjoining edges. The installer should take care not to damage or puncture the membrane during or after placement to maintain its integrity.

5.6 Pavement Recommendations

General recommendations for geotechnical site preparation and earthwork construction are provided in the sections below. In this section, we provide site- and project-specific recommendations and commentary for design and construction of proposed pavement areas.

5.6.1 Pavement Design Considerations

The site soil conditions are considered typical for asphaltic pavement section support. We recommend the standard City of Bellingham Pavement Section criteria for the road classification be applied for new public roadways. For private, light duty access roads and driveways, we recommend the following minimum asphaltic pavement section:

<u>Light Use Private Areas*</u>

Asphalt (HMA Class B)	3”
Gravel Base (CSTC/CSBC or Gravel Borrow)	6”

* For private roads required to allow heavy service vehicles or emergency vehicles, a 12-inch minimum total pavement section is recommended.

These sections are intended only as guidelines for design. Sections should be verified as suitable for the final development plans and adjusted if needed by the design engineer.

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5.6.2 Pavement Construction

Preparations for new pavement and exterior flatwork areas should generally follow the subgrade preparation recommendations in Section 5.8 and typical industry practices. Given the extent of the project area and the range of conditions observed, some variation in stripping depth should be anticipated to reach suitable subgrade conditions.

Subgrade for new pavement sections and flatworks should consist of generally stiff or medium dense native soils, or compacted approved fill installed over suitable native subgrade. Shallow subgrades will generally consist of silty sands and sandy silts of varying content. Granular subgrades should be lightly recompacted to establish a suitably uniform and medium dense state. Fine-grained subgrades should be prepared with a smooth finishing bucket to limit disturbance.
It is important to carefully assess pavement subgrades for suitability. Subgrade assessment should be done by a qualified geotechnical professional. We also highly advise conducting proof rolls of pavement subgrades, as proof rolling is well suited to identifying areas of problematic (weak) subgrade when under traffic loading. Any yielding or pumping areas identified should be over-excavated to remove under-performing subgrades and backfilled with gravel base material.

In cases where pavement subgrade is marginally suitable and additional excavation is not viable, or does not reach improved conditions within a reasonable depth, a geotechnical professional can assess the need for a minimum excavation depth for stabilization. Measures to stabilize poor subgrades will typically include specifying a certain structural fill replacement to "bridge" the weak conditions at depth, and/or placement of a ground fabric or geotextile for separation/structural purposes. The type and specification of subgrade reinforcement should be determined per the conditions at a given location. For situations requiring a lesser level of stabilization, a separation and filtration fabric may be sufficient (such as Mirafi 140N or 160N fabric). For heavier uses, an extruded polypropylene biaxial geogrid (i.e. Tensar BX series or similar) is recommended.

5.7 Stormwater Infiltration

Samples of outwash-type soils were collected from several explorations in the northwest and west-central areas of the site, and analyzed for grain size distribution with results as summarized above (Section 3.2.2); complete lab testing reports are attached in Appendix II. Saturated hydraulic conductivities (K_{sat}), representing infiltration rates, were then estimated using the Washington Department of Ecology Stormwater Management Manual (DOE SWMMWW, 2019) grain size analysis method. Rate calculations were performed using the grain size distribution data from lab testing (D10, D60, D90, and % Fines values). These variables were input into the following equation as adapted from Massmann, 2003 and Massmann et al., 2003:

$$log_{10}(K_{sat}) = -1.57 + 1.9D_{10} + 0.015D_{60} - 0.013D_{90} - 2.08f_{fines}$$

$$K_{sat}design = K_{sat}initial \times CF_t \times CF_v \times CF_m$$

Correction factors in the second equation were used to translate initial Ksat value to a corrected Ksat. We applied typical correction factors of 0.40 (CF_t) for grain-size test method and 0.9 (CF_m) for degree of influent control. A general value of 0.5 (CF_v) for site variability was applied to account for level of variation in fines content and consistency/density of the soils as observed, which may not

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be fully reflected in the samples analyzed. The total correction factor applied was CF_T = 0.18. Laboratory inputs and corrected Ksat values per sample location are presented in Table 5:

Table 5: Infiltration rate calculation results (Massmann Grain Size Method)

Loc.	Depth (ft bgs)	Class.	D10	D60	D90	Fines %	Ksat (in/hr)	Corrected Ksat (in/hr)
TP13	**4.0**	GP	0.64	12.22	27.16	1.5%	395	**71.1**
TP16	**3.0**	GP-GM	0.26	11.78	38.90	5.3%	43	**7.8**
TP24	**4.0**	SP-SM	0.11	4.94	25.59	8.3%	23	**4.1**

The samples analyzed were selected to represent the range of variability in the local outwash deposits observed in portions of the site. Generally, these granular soils contained fines contents in the range of 5% to 20%, and typically around 10% or less. The level of fines most directly affects the calculated Ksat value. Samples from TP13 (4 feet bgs) and TP16 (3 feet bgs) were chosen to represent gravel-rich soils at the low and moderate end of the average fines content spectrum, respectively. These soils found locally are highly transmissive and favorable for infiltration. The sample from TP-24 (4 feet bgs) better represents the sand-rich version of shallow outwash-type soils on site.

As expected, the gravel-rich samples with low fines yield a relatively high Corrected Ksat value which is subject to significant variation depending on local gravel and fines content. Whereas, the sandier deposits are typically more predictable for rate determination. For preliminary design purposes, we recommend design values not exceed the lower range of results. A ***Corrected Ksat of up to 4 inches/hour maximum is advised for use in preliminary design of select residential stormwater features with infiltration depths corresponding to outwash soils***.

We also reviewed the infiltration soil classification using the alternative USDA Classification System (MO5 Soil Technical Note 16; Benham et al., 2009) which is commonly used for prescriptive sizing of residential trench and drywell systems. The USDA method excludes the sample fraction retained on #10 sieve (gravel portion) and uses adjusted boundaries of sand sizes. The outwash soils sampled are classified as ***Coarse Sand*** per USDA textural criteria. Some notably sandier variations of the unit may be better classified as Sand. The designer may elect to use prescriptive design sizing for drywells based on DOE SMMWW (2019) standards. Actual soil conditions at the system location and depth should be reviewed to confirm final sizing criteria.

Samples of outwash soils from TP-13 and TP-24 were also tested for treatment-related properties. Samples recorded a **Cation Exchange Capacity (CEC) of 3.9 and 6.2 meq/100g** and an **Organic Content of 1.5% and 1.4%**, respectively**.** Organic Content values are found to exceed the 1.0% organic content requirements per the 2019 DOE SWMMWW. However, CEC values for native soils are near the 5.0 meq/100g minimum standards for CEC treatment requirements. Results are above or below the threshold corresponding to the local content of granular material, higher for sand and lower for gravel. If treatment is required, native soil amendment or import of an engineered treatment media may be necessary.

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Design Commentary

The tabulated (Table 5) preliminary design rates appear suitable for small-scale infiltration of rooftop stormwater where outwash conditions are present. We assume single residence systems would consist of prescriptive downspout infiltration features, either drywells where depths allow or shallow trenches where transmissive soils are depth-limited. Alternatively, a civil designer can be employed for engineered design of a lot-specific system.

Shallow soils at the northwest area entailing Lots 1 to 7 also appear to be suitable for pervious pavement use. Topsoil/subsoil in that area was observed to range from 1.5 to 3.0 feet thick. Below the thick cover soils, the subgrade was sandy soil. The above corrected design rate is suitable for pervious pavement design in this area. There may be similar opportunity for small, localized, stormwater systems servicing driveways, as well as pervious pavements/flatworks, that can be evaluated during individual lot designs at other locations. The current exploration data can be used as a general guide to identify potential infiltration areas. To confirm or adjust values for final design use, we recommend additional targeted explorations at specific locations/areas proposed for stormwater infiltration or pervious pavement use.

On the majority of lots where on-site infiltration and direct release is infeasible due to steep slopes, shallow bedrock, or other restrictions, stormwater should be collected and tight-lined to an approved dispersion location or to a community shoreline outfall pipe.

We recommend conditions be confirmed and systems be best fit on individual lots proposed for infiltration at the time of future lot development. The results of this feasibility-level review are suitable for general planning purposes, but are not intended to provide final design recommendations for individual lots without further review.

5.8 Earthwork and Excavations

5.8.1 General Site and Subgrade Preparation

We recommend stripping and removing topsoil, unsuitably soft or loose subgrades, uncontrolled fills, and soils containing organic remains or other deleterious materials. Stripping should include all proposed structure and pavement/flatwork improvement areas, and areas receiving structural fills to raise grade below or proximal to structures and pavements.

Once subgrade level is reached and any remaining unsuitable materials are removed, granular subgrades should be recompacted to a suitably dense, uniform, unyielding condition. We recommend subgrades beneath structures and pavements be evaluated by a geotechnical professional by appropriate means including T-probing and visual assessment to confirm competent unyielding conditions are established. Where unsuitable soils are identified, additional stripping or over-excavation and replacement with structural fill should be conducted under guidance of the geotechnical consultant.

A proof roll should be conducted over prepared subgrade with a loaded single-axle dump truck or water truck, or other appropriately sized and loaded equipment, under observation of a geotechnical professional. When access is not feasible, or weather conditions do not permit a proof roll, alternative means can be used to verify subgrade adequacy at the discretion of the geotechnical

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consultant. If areas of excessive deflection/rutting, looseness, or pumping are identified by proof roll, mark locations for rectification. Loose or rutting areas can be recompacted, subject to suitable moisture conditions, then re-assessed for suitability. Any pumping locations or persisting loose/soft areas likely reflect excessive moisture conditions and should be over-excavated until reaching suitable support conditions (or alternatively stabilized as directed by the geotechnical professional), then backfilled with new imported structural fill to restore planned subgrade level.

For over-excavations below structural loads, the width of excavation at base level is recommended to extend a 1H:1V distance outside of the loaded location corresponding to the depth of over-excavation. For instance, an over-excavation of 1 foot should also extend 1 foot in each direction from the edge of a structural load.

5.8.2 Difficulty of Excavations

The native soil conditions encountered at shallow levels (within a few feet of the surface) are anticipated to be viable for excavation and site preparations using traditional mechanical equipment (such as excavators/backhoes, bulldozers). Tooth-edge buckets may be preferable for excavation of dense or cemented materials as encountered. Flat-edged buckets should be used when preparing fine-grained subgrades to lessen disturbance of the subgrade, and when trimming excavation bases to final foundation design grade.

The depth to bedrock is found to vary within the project area, and in some cases is notably shallow. It is likely that conflicts with bedrock will arise when constructing infrastructure. Chuckanut Formation bedrock can typically be excavated with difficulty for road grading and utility trenching using standard equipment and mechanical rock-breaking equipment. Blasting is not recommended due to the potential for blasting to impact stability of adjacent sloping areas.

5.8.3 Wet Season Construction

Shallow native soils at the project site consist of silty sand to sandy silt with elevated fines content. These types of soil are highly moisture sensitive, and prone to significant issues such as weakening and degradation as a result of exposure to wet weather in the presence of construction traffic and activities. Furthermore, earthwork activities on moisture sensitive conditions can be difficult with additional costs and time commonly incurred for wet weather construction. Moisture-sensitive soils can be difficult to work and manage even in the dry season during periods of inclement weather. Finally, we recommend against placing frozen soil as fill, and against placing fill over frozen subgrade. Therefore, it is preferable to perform major earthwork construction for this project in the drier/warmer part of the year (late spring to early fall), and to avoid major grading activities during wet weather as possible.

For project earthwork activities that take place in the winter season or in inclement weather, we recommend the following guidelines:

- Limit machine and truck traffic on exposed subgrades to only as necessary. If traffic through an area is unavoidable, consider capping with temporary stabilizing material and/or leaving stripped levels high to be trimmed to grade later.
- Be prepared to substitute native material use (if planned) with imported structural fill. Be prepared to change imported materials to a low-fines content free-draining aggregate or clear rock substitute if moisture cannot be adequately controlled.

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- Grade subgrades for runoff, and provide outlets or dewatering for confined excavations that are susceptible to water inundation from runoff or seepage.
- Implement controls to the extent possible to limit surface runoff from adjacent areas from entering the excavation or work area.
- Avoid directing temporary runoff or water diversions from excavations onto nearby steeply sloping grades.
- Plan and conduct work in stages to minimize open time for sensitive subgrades. Preferably, strip and cover moisture-prone subgrades quickly if working in rainy weather.

5.8.4 Excavation Dewatering

Shallow conditions were generally free of wetness in the summer season, as seen in the test pit exploration logs. However, perched groundwater was observed locally, and shallow restrictive conditions are commonly present. This indicates a potential for seasonally induced seepage and water transmission through the shallow subsurface. While development of a full perched water table is unlikely given the sloping grades of the site, migration of shallow transient water from uphill sources into excavations may be expected to occur in the winter and spring seasons. Perched water may collect locally in topographically convergent areas.

Dewatering actions may be needed to maintain workable shallow excavations if site preparation or utility work is done in the wet season or under sustained wet weather. We anticipate conventional methods should be sufficient for controlling transient water inundation, including pumping for evacuation and providing temporary runoff outlets from work areas. Some additional expense and difficulty should be anticipated for wet season site preparation and utility construction.

The scope of work completed to date has not included direct monitoring of groundwater fluctuations through the wet season, or characterization of flow rates/volumes for subsurface water transmission. A hydrogeologic study has not been conducted at this site. The information and commentary provided is intended only for planning purposes, and does not necessarily provide recommendations for dewatering design.

5.8.5 Excavation Shoring

In Washington State, shoring or sloping is required for excavations that are deeper than 4.0 feet (WAC 296-155, Part N). Excavations for this project are anticipated to be primarily shallow, although some work may call for depths in excess of 4 feet. If shoring is elected due to space constraints, or as the preferred method of construction, the system must be evaluated and designed by a registered professional engineer licensed with the State of Washington. The shoring designer should review the findings of this report, and account for potential loads including soil pressures (active or at-rest, as applicable), hydrostatic influences, and loads from sources such as adjacent stockpiles, heavy equipment, and traffic.

In addition to providing safe excavation access and egress in accordance with OSHA requirements, shoring should be designed to adequately protect adjacent features (such as existing utilities, structures, pavements) from detrimental effects including during installation and removal of the shoring. In the event that shoring is required in proximity to an existing feature/facility, we recommend the standards for protection be clearly established in project requirements. In some

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cases, an acceptable level of damage to adjacent conditions is suitable in order to expedite work. The standards for repair to existing features as a result of excavation shoring use should also be agreed upon prior to construction.

5.8.6 Temporary Cut-slopes

We recommend all temporary construction slopes adhere to local, state, and federal requirements. Establishment and maintenance of suitable cut-slopes to provide worker and site safety is the responsibility of the contractor. The following guidelines for cut-slope preparation are provided for general planning purposes only, and should be revised as necessary once conditions are open and observed during construction.

Temporary cut-slopes within the shallow native soils should be sloped no greater than 1:1 (H:V), corresponding generally to "Type B" soils. If soils are locally soft or loose with apparent instability, or if work proceeds in wet conditions, a down-grading of the soil type and corresponding reduction to 1.5:1 (H:V) or less is recommended. Excavations can be evaluated in construction by a qualified geotechnical professional to determine if steeper grades are permissible for short-term and/or relatively small slopes based on actual observed condition and soil strength.

Loads from external factors, including but not limited to heavy equipment, traffic, stored materials, and soil stockpiles should be avoided directly above unreinforced cut-slopes. If loading is unavoidable, a lesser slope angle or temporary shoring of the location may be necessary. We recommend cut-slopes that will remain open for an extended duration be protected from exposure to inclement weather conditions. Covering slopes with plastic can help prevent erosion and degradation of the slope face over time. If utilized, cover sheeting should be anchored sufficiently to resist wind displacement and overlapped to minimize leakage.

5.9 Structural Fill Recommendations

5.9.1 Use of Structural Fill

Structural fill constitutes all fill soils placed underneath structures or pavements for support. Additionally, soil backfills against foundations and walls, and soils used similarly for the purpose of providing lateral stability to structures, are considered structural fill.

In general, structural fill shall consist of primarily granular and non-plastic aggregate of suitable gradational characteristics, that is relatively uniform in mineral composition, contains no discernible organic materials, and is free of other trash and deleterious materials. It is typically recommended that all aggregate be less than about 4 inches in diameter, maximum particle size. For thin lifts or specific applications, a lesser maximum size may be required (maximum particle size of 2/3 lift thickness, or as specified for use).

We recommend structural fill be placed over suitably prepared and engineer-verified subgrade as recommended above. We advise against placing structural fills intended for building and pavement support over existing unverified uncontrolled fills, or unsuitable soft or loose subgrades, due to the elevated risk of settlement of underlying strata. In exceptions, fills may be placed as an approved subgrade stabilization measure under the evaluation and guidance of a geotechnical professional for an express location and purpose.

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5.9.2 Installation and Compaction

Structural fills should be properly moisture controlled or conditioned to within 3 percent of optimum moisture level for the specific material to encourage proper compaction. In the dry season, granular fills residing in stockpiles may be excessively dry and need to be wetted prior to or during use. In this event, it is advisable to proceed cautiously with water application until a moisture-conditioning program can be established. In the wet season, care should be taken to protect structural fill stockpiles from rainfall. Fills with excessive moisture levels must be removed and mixed, stored, or dried/aerated until within an acceptable range for use.

Installation of structural fill shall be done in horizontal lifts not exceeding about 8 to 10 inches maximum loose-thickness. Thin lifts will be needed for small machinery or hand-operated equipment in order to achieve compaction. Per *WSDOT Standard Specifications 2-03.3(14)* and our professional judgment, fills should be benched when placed on grades steeper than 3H:1V.

Structural fills shall be compacted with appropriately sized equipment to a uniformly dense and unyielding condition. For all fills placed beneath or as backfill for structures, we recommend a minimum 95% compaction be attained. A minimum compaction standard of 95% is also recommended for the upper 2.0 feet of pavement subgrades, as well as the upper 4.0 feet of utility trench backfill beneath paved areas. Beyond 2.0 feet below the base of pavement away from structures (4.0 feet at utility trenches), and for non-structural utility backfills (outside of paved areas only), a minimum 90% compaction is considered suitable. Compaction shall be based on the maximum dry density of the material, determined by laboratory testing per ASTM D-1557 test method. Field compaction testing shall be conducted as necessary to verify compaction of each lift. Compaction testing should be performed frequently as work begins to establish suitable placement/densification methods, then as needed to assure project standards are met.

5.9.3 Existing Material Suitability

On-site soils encountered in explorations consist predominantly of silty sand and locally sandy silt at shallow levels. Assuming construction in dry conditions, excavated non-organic native soils produced in cut areas are generally considered suitable for use as non-structural grading fills in landscaping areas, and as native material for trench backfill outside of the road prism (per *WSDOT SS 9-03.15*). That is, provided the material is of sufficient quality and condition to be compactable and meet other project requirements for the intended use.

Granular native soils may be suitable for use as subgrade-level fill below lightly loaded floor slabs and pavements. Site soils are moderately to highly moisture sensitive due to high fines content, and as such will only be suitable for reuse in dry weather. Native materials may need to be moisture-conditioned prior to placement. Native soils proposed for reuse on site should be stockpiled separately from unsuitable materials, and evaluated for suitability before installation by laboratory testing and/or visual means of approval. Additional testing and quality control efforts should be expected for use of native soils in comparison to imported fills.

5.9.4 Imported Material Specifications

Imported aggregate meeting plan requirements for the intended use, and the general recommendations of this report, is considered suitable for use as structural fill. For general-use structural fill, we recommend well-graded imported material meeting the specification for Gravel

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Borrow (*WSDOT SS 9-03.14(1))*. A performance equivalent may be approved for substitution by the project engineer and geotechnical consultant.

Gravel backfills placed behind retaining walls and retaining foundations must be free-draining, and shall comply with *WSDOT SS 9-03.12(2)* unless otherwise specified or approved by the wall design engineer. Free-draining materials have a typical maximum of around 3% fines content (depending on material type), and thus standard structural fill may not be viable for this purpose.

If work occurs during excessively wet weather, or if water is unavoidable within excavations, it may be preferable to substitute standard structural fill with a material not affected by water presence. For this purpose, a clear angular rock such as 1-1/4" clear ballast may be considered, subject to approval by the geotechnical consultant for the proposed use. If utilized, clear rock shall be installed as recommended above and compacted to an unshifting, unyielding, and uniformly dense condition as verified by visual methods and/or proof-roll.

Controlled-density fill (CDF) may be suitable for use in substitution for structural fill in some cases. If proposed, CDF use should be reviewed by the project engineer and geotechnical consultant before its placement.

Laboratory testing should be conducted in advance of construction to evaluate and verify the proposed imported materials are suitable for use. In the event that a material does not meet the project specification, the applicable engineer and geotechnical consultant may review the results for conditional acceptance. However, the contractor should also be prepared to find an acceptable alternative material if the initial source is unsuitable.

5.10 Utility Construction

5.10.1 Utility Trenching and Excavation

Trenching and excavations for utility improvements will typically encounter topsoil and shallow glacial deposits or colluvium (locally variable sand, silty sand, and sandy silt) through a few feet depth. Upper deposits are underlain at varying depth by cemented/densic glacial soils and bedrock of the Chuckanut Formation (Sandstone, Siltstone). We have made the following inferences based on conditions encountered:

- The native upper soils are considered moderately susceptible to raveling and sloughing on average. Actual degree will vary locally by soil type. Steep trench walls may be difficult to maintain for even shallow excavations. At minimum, a contingency plan for slope layback or temporary reinforcement should be in place, especially for trenching in limited space.
- If trench work is conducted during wet weather, seepage from perched water and soil saturation may increase the likelihood of trench wall raveling/sloughing.
- Due to the potential for shallow saturation and seepage as well as inundation from upgradient transient waters into confined excavations, trenching and utility work is generally not recommended to be done in the winter season.

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- Bedrock presence at shallow depth can significantly hinder the timing and progress of trenching preparations. Additional potholing is recommended to be done during construction for pre-planning purposes as the project advances.

The longitudinal extent of trenching should be kept to short intervals or segments, with pipe installation and back filling completed prior to opening new trench sections. This will limit the length of exposure time to trench wall drying or rain-wetting with the consequent sloughing that may be expected with exposure time.

It is the responsibility of the contractor to establish a safe and secure work environment for entry and work performed in utility trenches. The recommendations in the Earthwork and Excavations section of this report should be followed, as well as any state and federal safety regulations. The contractor is also responsible for monitoring the condition and safety of excavations including utility trenches over the open time. In the event of instability or signs thereof, the contractor should be prepared to modify the excavation to a more stable configuration (by using or reducing cut-slopes) or utilize temporary shoring. It shall be understood that conditions can change and local variations can occur. The above guidance is intended for general planning of trench work, and does not represent a guarantee of conditions or the success of specific approaches. Any significant variation from the above encountered during construction should be reassessed by a qualified geotechnical professional.

5.10.2 Backfill and Pipe Zone Bedding

Typical trench and pipe backfilling practices are considered appropriate for this project. As is noted above, some materials excavated during trenching for this project may be suitable as replacement trench backfill in select areas. The material should be evaluated for its suitability upon excavation but before it is planned for reuse. The following recommendations are provided for trench back fill and pipe zone bedding considerations.

- Imported gravel for pipe zone bedding should consist of aggregate material satisfying the specification requirements of *WSDOT 2018 Standard Specifications 9-03.12(3)*.
- Unless otherwise specified by project or local municipal utility requirements, imported gravel for trench backfill below roadways and beneath paved areas should at minimum meet the specification requirements of *WSDOT 2018 Standard Specifications 9-03.19*. If allowed, trench backfill outside of paved and trafficked areas may consist of suitable native or other non-structural material (per *WSDOT SS 9-03.15*).
- Based on the interpreted suitability of native subgrades at likely utility trench depths, it will not be necessary to use an additional foundation layer when constructing utilities at the project site.
- To limit potential future settlement of pavement sections above newly installed utilities, compact the pipe bedding zone material to not less than 95% of its maximum dry density. If a “self-compacting” material is used (such as pea gravel), the material should be well distributed and tamped as needed to achieve an unyielding condition before backfilling.
- For trench backfill below pavements, it is preferable that the level of compaction achieved is at least 97% (no less than 95% standard minimum). However, the pipe manufacturer’s

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specifications for compaction of materials adjacent and above the pipe should be observed to prevent possible damage to the pipe and any connections.

We recommend against using alternative soil densification measures such as jetting or flooding as a substitute for proper mechanical backfill compaction. Utility backfills and compaction procedures should adhere to the recommendations provided in this report for Structural Fill.

Where lateral thrust blocks are to be constructed to provide lateral pipe restraint, the concrete should be cast neat to undisturbed trench wall soils to ensure that adequate lateral load support is provided by the in-situ soils. Backfill placement for support of thrust blocks is not recommended.

5.11 Contractor Responsibilities

Some variability in substrate composition should be anticipated across the study area. It is not plausible or reasonable to expect that a pre-construction investigation will identify all variations at a site, nor does the exploration program executed for the purpose of this study constitute a complete and exhaustive survey of site subsurface conditions. A reasonable level of extrapolation has been applied to the interpretations and conclusions of this report. The contractor is responsible for reviewing this information in full, and asking for clarifications, if necessary, prior to conducting work. The contractor should also conduct independent confirmation of conditions as needed to successfully plan and implement their proposed systems of construction, including but not limited to shoring and dewatering design, if required. If the opportunity to conduct additional evaluation is presented and waived by the contractor, neither the client nor Element Solutions shall be held liable for data limitations in design of construction systems and methods.

In all instances where unusual or unanticipated subsurface conditions are encountered during any stage of the site preparation or construction process, it is the responsibility of the construction contractor to notify the client and/or the engineering design team. The project team should then be prepared to provide on-site geotechnical supervision prior to further excavation, grading, or construction. Due to the compositional variability observed in shallow soils across the site and the potential for excavation and trench caving, a geotechnical engineering professional should be consulted as needed during all temporary excavations to confirm soils and excavation/trenching conditions.

All on-site soil excavation and stockpiling should be performed in accordance with industry-standard best practices and protected from erosion in a manner consistent with the approved Temporary Erosion and Sediment Control (TESC) Plan. The contractor is responsible for implementing and maintaining erosion control procedures and devices in accordance with local and state requirements.

5.12 General Critical Area Guidelines & Recommendations

The following guidelines and recommendations are intended to minimize the impacts and inherent risks associated with development within or in proximity to geologically sensitive critical areas. The information is site- and project-specific based on our understanding of the proposed development and existing conditions at this time.

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5.12.1 Stormwater Management

Development drainage features and stormwater controls should be implemented in a manner that does not lead to an increased potential for erosion or instability on the site slopes, nor places downgradient properties at risk. Generally speaking, ***we recommend that all stormwater from new impervious surfaces be captured and managed***. On-site stormwater release systems (infiltration or dispersion) for lots or roadways are not considered viable among areas on or proximally above steeply sloping topography. With exception of localized lot-scale infiltration at areas of the property fronting Viewcrest Road, and possibly pervious pavement driveways at some other lots to be determined, the site is generally considered infeasible for infiltration. The combination of small lot sizes and sloping topography also appears to limit use of individual lot dispersion systems within most of the building lots.

Project discussions indicate the primary stormwater management for the site roadways will employ subsurface storage volume (i.e. vaults, large pipes, stormtech units, etc.) for flow control. One option under consideration for disposal is to collect and route stormwater to the eastern part of the site, then convey it downhill to the southeast via a big outfall pipe for release at the coastline (above marine water level). In our opinion, this is a viable course of action from a geotechnical and geohazard protection perspective, assuming the downslope tightline is properly sited and constructed to minimize risk of failure.

A second option, which may help to avoid construction of a large outfall pipe down the steep coastal slope, is to employ upland dispersion at select areas. Dispersion is considered among forested open-space areas of relatively lower gradient topography downhill of the main development area. In our opinion, selective dispersion is also a viable strategy provided the systems are preferentially sited and adequately designed/built so that stormwater is discharged over a sufficiently large area.

Based on the findings of this study, we conclude and recommend the following criteria for proper management of new stormwater generated by lot and roadway development:

- Infiltrate stormwater only where conditions are proven to meet municipal feasibility criteria, and steep slopes are not present or in proximity. Additional lot-scale review to confirm infiltration suitability with respect to final development plans is advised.
- Dispersion or down-gradient release of collected stormwater within individual lots is generally <u>not advised</u>. Underlying properties and slope areas could be negatively affected by release of stormwater.
    - Possible exceptions include lots along the southeast perimeter of the development that contain areas of gentle downslope topography (see below).
    - Depending on final development layout, there may be other exceptions of lots viable for localized dispersion. We recommend reviewing individual lot dispersion on a per-case basis, in the context of final layout and surrounding conditions, if considered for use.
- Dispersion of collected lot and/or roadway stormwater can be considered among downhill forested areas of the site. For on-site dispersion, we recommend:
    - Divide dispersion to utilize several areas so that stormwater release is not excessive at any one area, and for ease of design/construction among variable grades.

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- Employ systems which control and disperse outflow over a wide area (such as a trench with level-spreader). Do not use point-source outflows in upland areas.
- Disperse among areas with lesser grades and adequate vegetation.
    - We recommend limiting dispersion to areas around 30% grade or less.
    - Avoid or minimize clearing of forest vegetation, including trees and undergrowth, around and downhill from dispersion locations.
- A minimum setback of 100 feet is recommended for engineered dispersion above the southeast coastal bluff slope.
- Based on these guidelines, areas with potential suitability for communal dispersion may include:
    - Lower gradient slope areas along the bottom of Lots 28, 29, and 30 to 32, as well as the bordering upland part of "Open Space Tract A" outside of the recommended setback.
    - Gentle mid-slope area of Lot 33, lower half of Lot 34, and adjacent ROW (to be vacated).
    - Area along east borders of Lot 35 and 36 (drains towards wetland zone).
- Element Solutions should be retained to consult on the placement and design of on-site dispersion systems, if incorporated. ES can assist in identifying optimal locations, and perform field reconnaissance for verification of suitability at proposed dispersion areas.

- All stormwater from roof runoff, pavements, and exterior drains should be tightlined from the collection points to a lot catch basin, then directed to a conveyance tightline leading to the approved dispersion facility or outlet point.
- Foundation and wall drains should be conveyed separately from other drain sources, or adjoined at a suitable down-gradient location, to prevent the backflow of water to footing drains. Given the low volume of these features, it is commonly permissible to outlet footing or wall drains at a suitably gentle and vegetated area away from the structure.
- Stormwater from upland and neighboring sources should also be properly controlled by the adjacent (off-site) properties. If necessary, construction of the project should also implement safeguards at its boundaries to lessen the potential for overland flow from entering the property. This may include incorporation of small swales, yard drains or perimeter drain systems to maintain a dry site.
- All above-grade tightlines should be composed of sturdy rigid material resistant to damage (such as PVC or welded HDPE pipe), sized adequately for the anticipated outfall volume, and anchored sufficiently to the ground to minimize the potential for damage and failure. Tightlines should be inspected periodically, and repaired or replaced as needed to maintain a safe working condition. For directed outfalls, appropriate energy reducing features should be used at the release point as necessary to minimize erosion. Examples include a perforated T-stub/spreader pipe, rock pad, or release onto exposed bedrock.

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5.12.2 Site Management During Construction

Additional care is necessary when construction occurs on or near steep grades. For the purposes of critical area protection and erosion management, grades of 30% or over are subject to regulation under City of Bellingham Code. The following guidelines and recommendations pertain to regulated slope areas.

- Outside of structural areas, new fills on slopes should be minimized (other than as needed to backfill ancillary areas around footings, and below hardscapes). Fills placed on a slope face outside the confines of a structure add weight to the slope, and may increase the risk of instability or erosion.
- Temporary stockpiling of excavated material or fills, or storage of heavy construction materials and machinery, shall be avoided on sloping areas. Stockpile soils for import/export at the lowest gradient area available pending transport or use.
- Construction practices shall take care to disturb or impact as little area as possible. Impacted areas should be restored with top-dressing and appropriate plantings for the environment following construction. Avoid disturbance outside of the established development boundaries on each lot.
- Temporary erosion controls:
  - Systems and procedures should be put into place as appropriate for the site, project, and timeframe/season of construction. TESC measures should include downslope and sideslope clearing/disturbance limit barriers or demarcations.
  - During periods of major excavation and during benching or excavation of rock on or near sloping grades, additional downslope safeguards should be installed as needed to prevent soil and rock fall from leaving the site.
  - The contractor is responsible for implementing and maintaining TESC throughout earthwork activities, and for working within accepted project limits to avoid unnecessary impacts to adjacent areas (especially critical areas).

5.12.3 Long-term Erosion Control and Maintenance

For long-term site care and management of critical area slopes:

- We recommend goals of low impact or vegetative enhancement be adopted for exterior areas outside building and road development zones, including preservation of existing trees and brush where possible. This will help minimize the chances of future instability on sloping areas following development. We advise planting of appropriate brushy vegetation among ancillary areas near structures and roads that are unavoidably disturbed during construction, either at the end of construction or in the future under final ownership.
- Removal of mature trees on steep grades should be limited to only those directly necessary to construct the project. If select trees are a concern for current or future hazard to structures or roads, a qualified arborist should be consulted to evaluate tree-limbing, topping or removal. Full removal actions should also be reviewed by a licensed geologist where in conflict with critical area slopes, and may require mitigative measures.

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- Promoting future growth of strong-rooting brushy plants and new trees is encouraged both following construction and in the long term. Thick and healthy vegetation will assist in retaining cover soils, increase the hydrologic resistance of surface conditions, and lessen the risk of erosion that could result from incidental surface runoff or other overland drainage issues that could arise.
- Major landscaping alterations should be avoided on slopes outside of planned development areas unless properly reviewed by a geotechnical professional and found to be suitable for the location and surrounding conditions. We generally advise against placement of significant fills or terracing alterations on slopes, which could affect the downslope conditions or result in instability.
- If conditions are observed to evolve or deteriorate in the future and pose a potential concern for stability of the site or adjacent areas, we recommend conditions be re-observed at that time. Element Solutions should be contacted to reassess the site conditions, and can provide guidance for stabilization and best management practices at request of the property owner.

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6 Closure

Thank you for the opportunity to contribute our expertise to your project. Please do not hesitate to contact us at (360) 671-9172 if you have any questions or comments regarding this report.

Sincerely,

Ryan Cooper, GIT
Project Scientist

November 3, 2021

John R. Gillaspy

John Gillaspy, LEG
Environmental Services Manager

November 3, 2021

Lorne Balanko, PE
Senior Geotechnical Engineer

Statement of Limitations

This document has been prepared by Element Solutions for exclusive use and benefit of the Client. No other party is entitled to rely on any of the conclusions, data, opinions, or any other information contained herein. This document represents Element Solution's best professional judgment based on the information available at the time of its completion and as appropriate for the project scope of work. Services performed in developing the content of this document have been conducted in a manner consistent with that level and skill ordinarily exercised by members of the geologic engineering profession currently practicing under similar conditions. No warranty, expressed or implied, is made.

Exploration logs presented in this report represent locations and dates of field work. Conditions encountered by location may not be fully representative for other areas of the project site, and may vary depending on the timeframe of exploration. A degree of natural variation should be anticipated within native subsurface conditions; greater variation is likely where previously altered conditions or uncontrolled fills are found. If conditions are present in construction that are different than those encountered in this study, Element Solutions should be contacted to provide review and consultation, and to reevaluate our recommendations if necessary. We also recommend review of final plans and specifications by Element Solutions, as well as changes to the project scope that could impact the intent of our recommendations.

If the client elects to retain another geotechnical consultant for additional work or construction phase geotechnical support, the retained firm or individual is expected to review this report in full. They shall either verify and agree with the interpretations and recommendations provided, or offer their own recommendations. Element Solutions shall not be responsible for revised interpretations or recommendations made by others.

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References

1) Lapen, T.J. Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington. Washington State Department of Natural Resources, Division of Geology and Earth Resources Open File Report 2000-5, December 2000.
2) Natural Resources Conservation Service, Web Soil Survey, U.S. Department of Agriculture, http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx, accessed online July, 2020.
3) Palmer, S.P., Magsino, S.L., Bilderback, E.L., Poelstra, J.L., Folger, D.S., and Niggemann, R.A. Liquefaction Susceptibility Map of Whatcom County, Washington. Washington State Department of Natural Resources, Division of Geology and Earth Resources Open File Report 2004-20, September 2000.
4) Wyllie, D. C., and Mah, C.W. Rock Slope Engineering – Civil & Mining. 4th Edition, 2005. Taylor & Francis e-Library publication.

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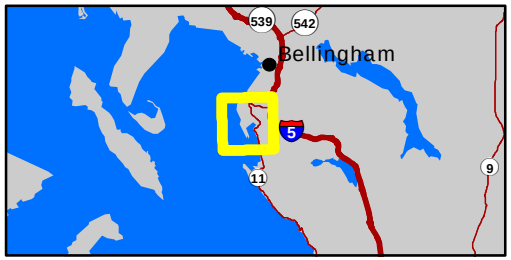
Appendix I

1) Figure 1 – 1:24,000-Scale Site Vicinity Map, Jones-Edgemoor Property, Bellingham, WA
2) Figure 2 – Project Area & Lot Layout Overview Map, Jones-Edgemoor Property, Bellingham, WA
3) Figure 3a – Topographic LiDAR Map with Percent Slope Shading

   Figure 3b – Project Lot Layout Map with Percent Slope Shading
4) Figure 4 – Project Overview LiDAR Map with Major LHA Features Annotated
5) Figure 5 – Detail LiDAR Map of Northeast Landslide Hazard Area and Buffer.



Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018, WA DNR 2016
[Imagery]USDA NAIP 2017
[Lidar] COB 2013, USGS, QSI 2016

Study Area

Highways

Roads

RailRoads

Feet
0 800 1,600 3,200 4,800

1:24,000

1 inch = 2,000 feet

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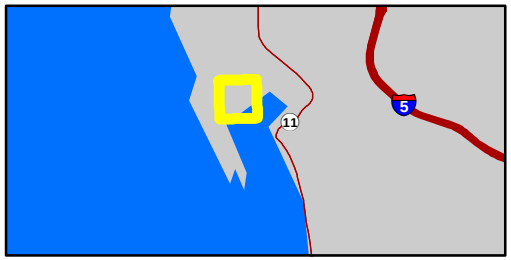
Figure 1

Jones - Edgemoor Estate
Site Vicinity Map

Date: 7/21/2021

P:\Pse Project\2020094\ENVRMNT\GIS\Figure 1 - Site Vicinity Map.mxd



Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Imagery] COB 2019

Study Area
Roads
Lot Layout

Feet
0 250 500

1:3,000

1 inch = 250 feet

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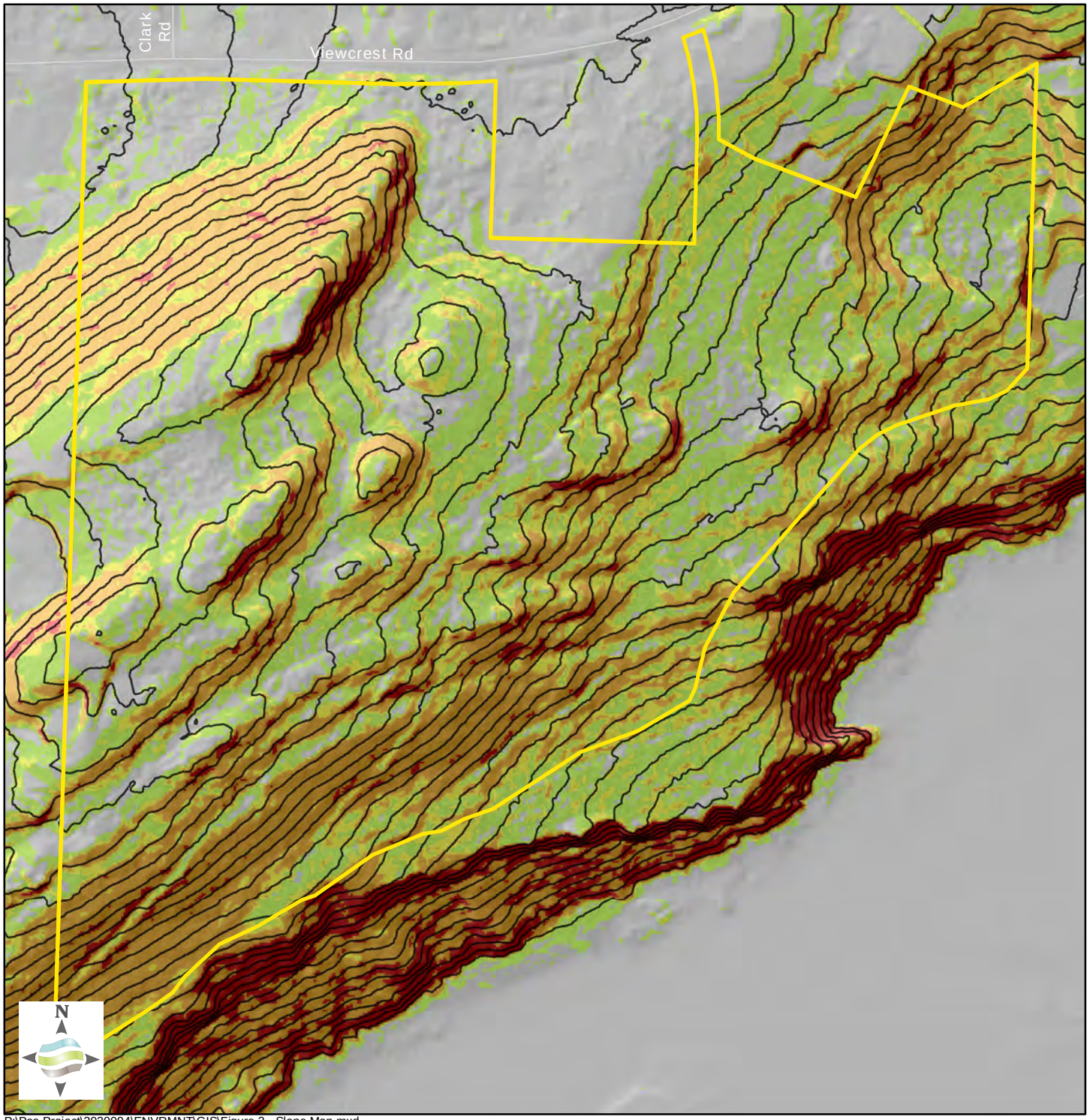
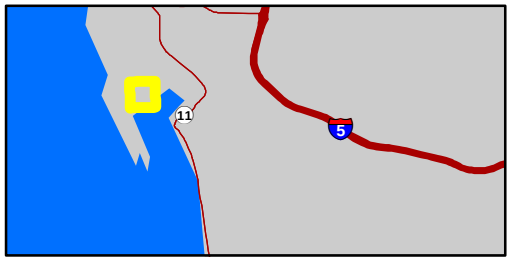
Figure 2

Jones - Edgemoor Estate
Project Area and Lot Layout

Date: 2/25/2022

P:\Pse Project\2020094\ENVRMNT\GIS\Figure 2 - Project Area Site Map.mxd





Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Lidar] COB 2013

Study Area
10ft-Contours
Roads

Percent Slope
- 0 - 15
- 15 - 30
- 30 - 40
- 40 - 80
- 80+

Feet
0 250 500

1:2,400

1 inch = 200 feet

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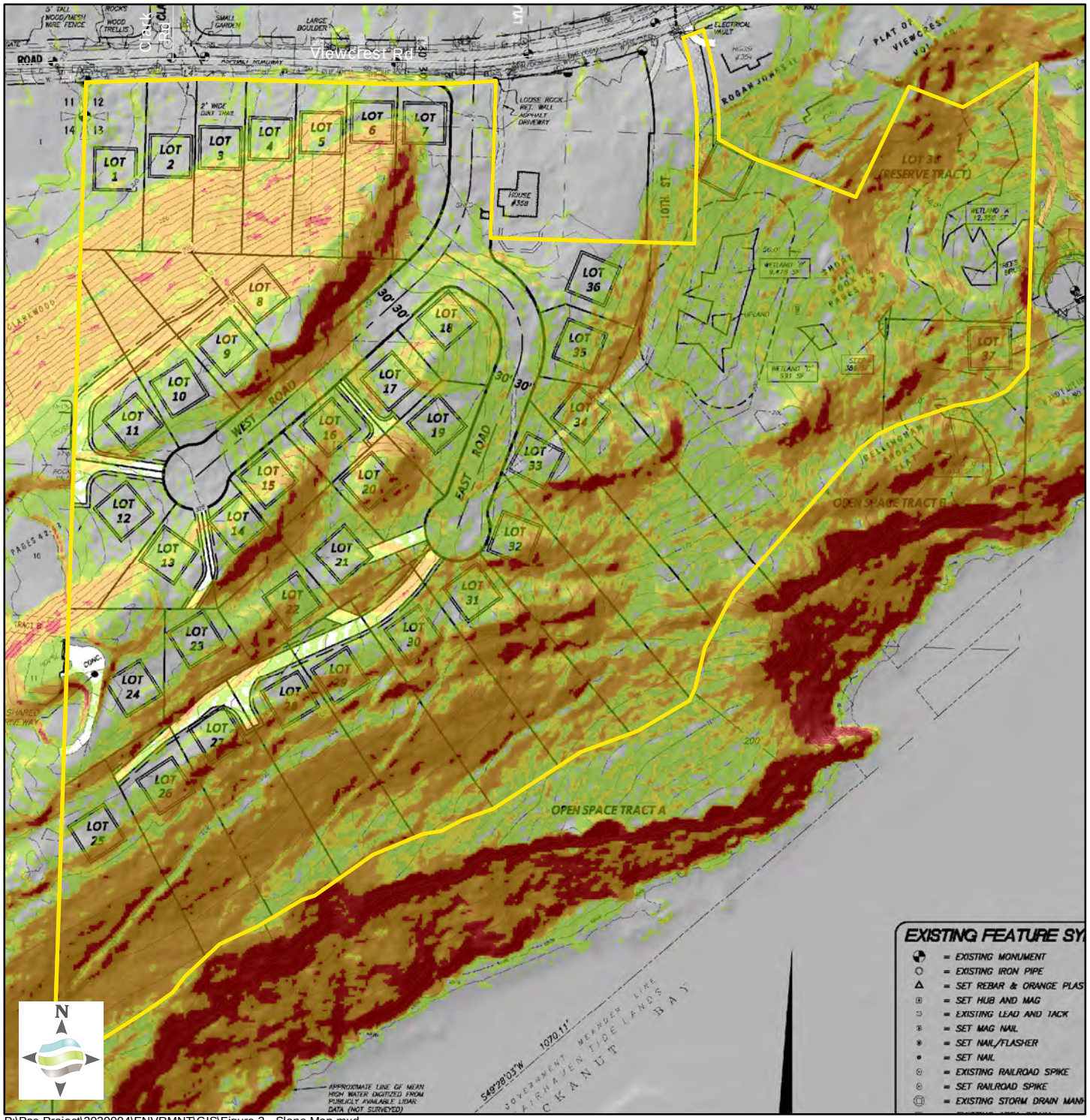
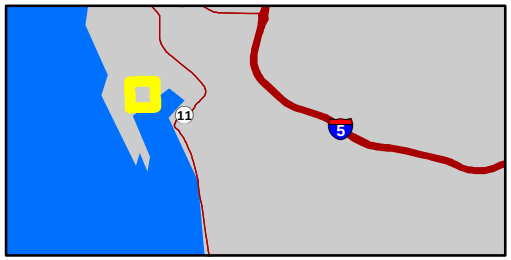
Figure 3A

Jones - Edgemoor Estate
Percent Slope Map

Date: 2/25/2022

P:\Pse Project\2020094\ENVRMNT\GIS\Figure 3 - Slope Map.mxd





Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Lidar] COB 2013

Study Area
Roads

Percent Slope

- 0 - 15
- 15 - 30
- 30 - 40
- 40 - 80
- 80+

Feet
0 250 500

1:2,400

1 inch = 200 feet

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Figure 3B

Jones - Edgemoor Estate
Percent Slope Map

Date: 2/25/2022

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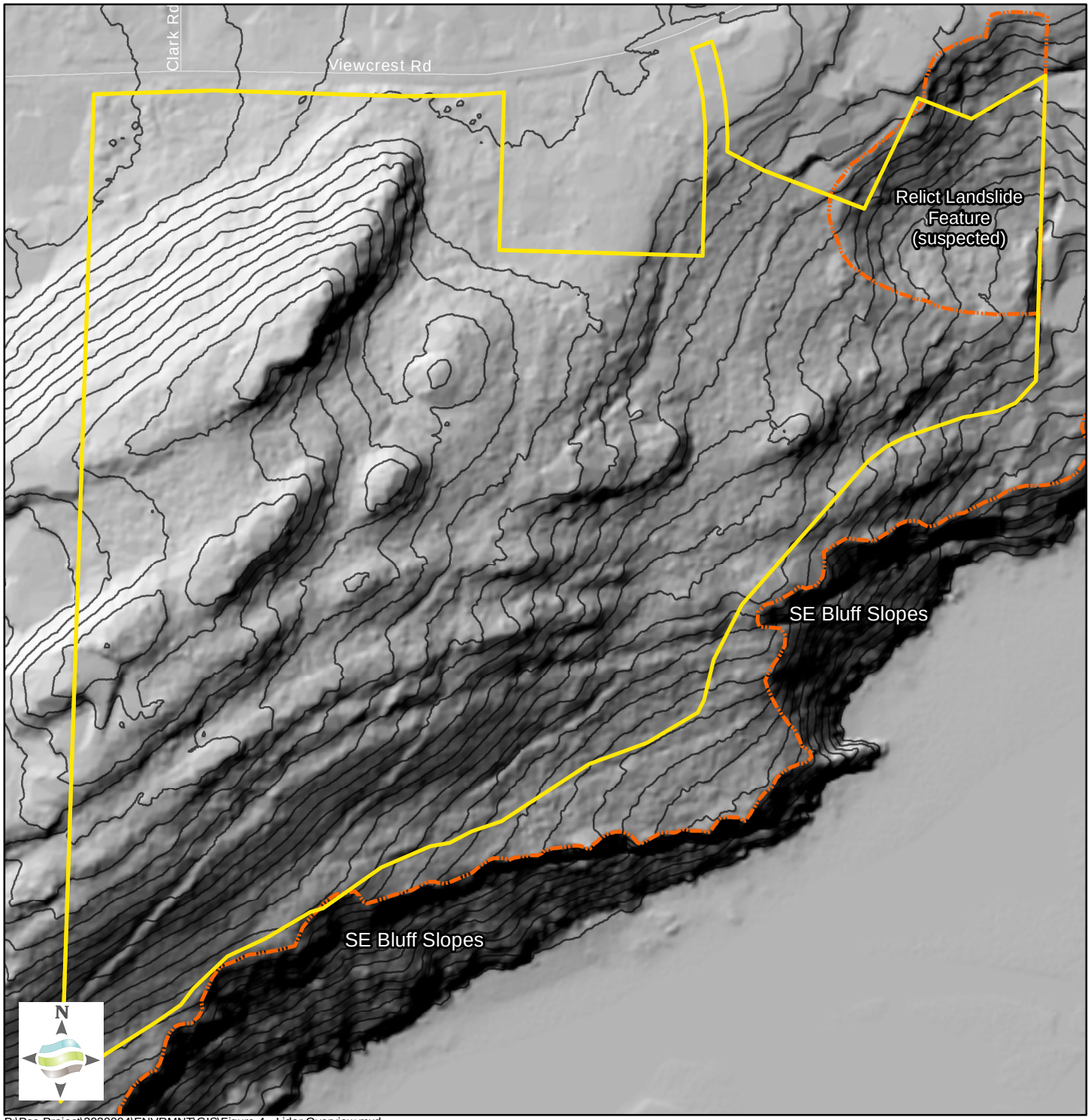

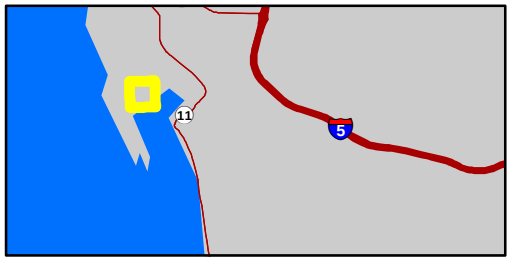

Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Lidar] COB 2013

Study Area
Landslide Hazard Area (LHA)
10ft-Contours
Roads

Feet
0 250 500

1:2,400

1 inch = 200 feet

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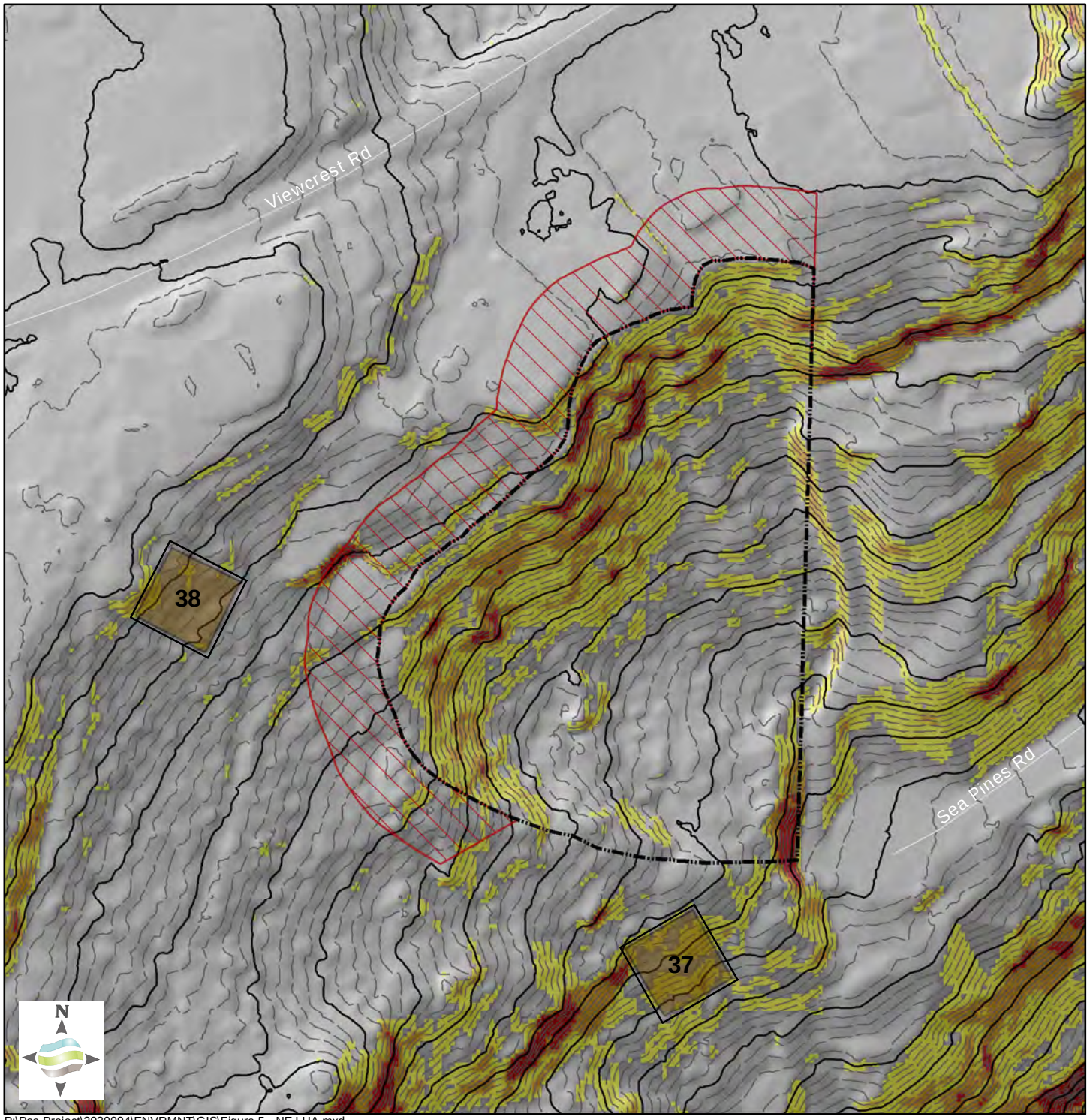
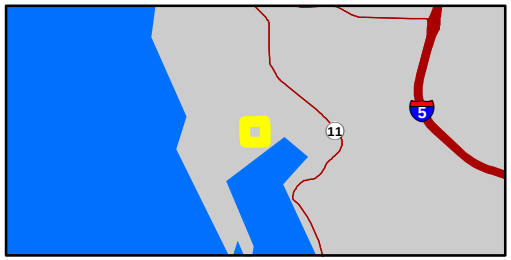
Figure 4

Jones - Edgemoor Estate
LiDAR Overview w/ Feature Annotation

Date: 2/25/2022

P:\Pse Project\2020094\ENVRMNT\GIS\Figure 4 - Lidar Overview.mxd





Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Lidar] COB 2013

Study Area
Landslide Hazard Area (LHA)
50ft LHA Buffer
Building Areas
10ft-Contours
2ft-Contours

Percent Slope

0 - 40
40 - 60
60 - 80
80+

Feet
0 120 240

1:1,200

1 inch = 100 feet

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Figure 5

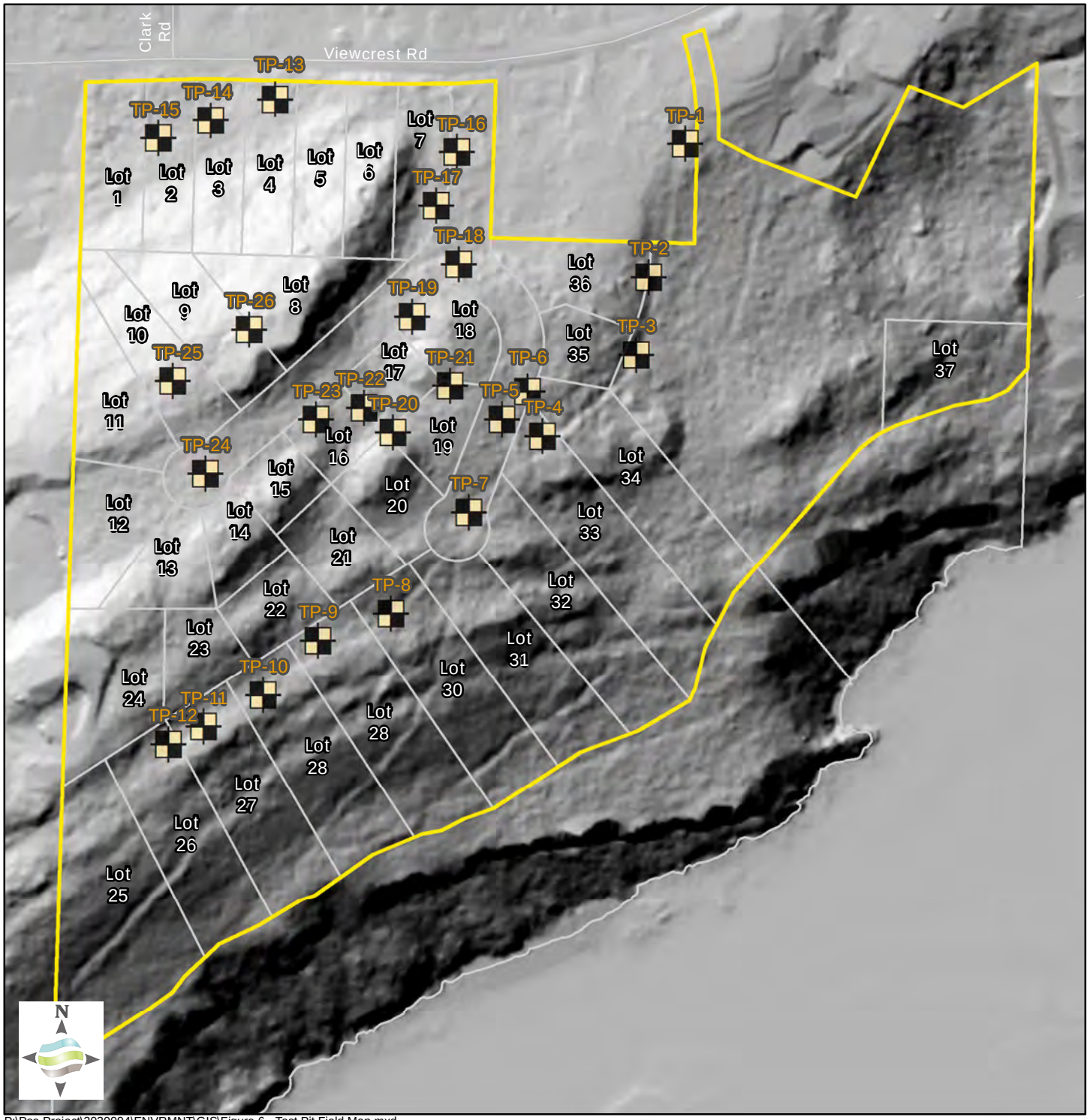
Jones - Edgemoor Estate
NE Landslide Hazard Area

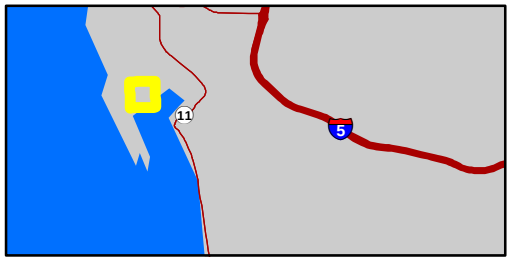
Date: 8/2/2021

P:\Pse Project\2020094\ENVRMNT\GIS\Figure 5 - NE LHA.mxd

Appendix II

1) Figure 6 – Project Map with Test Pit Locations
2) Test Pit Logs, TP1 to TP26 – June 30 and July 1, 2020
3) Laboratory Testing Reports, GeoTest Services Inc., Project No. 20-0587. July 16, 2020
4) Laboratory Testing Report, Northwest Agricultural Consultants, Report No. 52022-1-1. July 21, 2020
5) Exhibit A – Field Photos of Exploration Conditions, June 30 and July 1, 2020
6) Figure 7 – Project Map with Measured Depths to Bedrock by TP Location
7) Figure 8 – Sea Pines Work Area Map with Test Pit & Hand Auger Locations
8) Exploration Logs – Sea Pines Area, TP1 to TP2, HA-1 to HA-2 – November 13, 2020
9) Exhibit B – Field Photos of Sea Pines Site Conditions & Explorations, November 13, 2020





Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Lidar] COB 2013

Test Pits

Study Area

Lot Layout

Roads

Feet

0 250 500

1:2,400

1 inch = 200 feet

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Figure 6

Jones - Edgemoor Estate
Test Pit Field Map

Date: 2/25/2022

P:\Pse Project\2020094\ENVRMNT\GIS\Figure 6 - Test Pit Field Map.mxd

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TEST PIT NUMBER TP1

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 252' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; damp; root material present. **[Topsoil]**	
				0.7	251.3
		ML		(ML) SILT WITH SAND; ~50-60% fines; dark grayish brown; soft to medium stiff; cohesive; low to non-plastic; damp; dark orange oxidation staining ~3-3.5'; chunks of asphalt present. **[Uncontrolled Fill]**	
2.5					
				3.5	248.5
		SM		(SM) SILTY SAND, some gravel and cobbles; ~30-40% fines; tan to light grayish brown; medium dense, increasing with depth; low cohesion; non-plastic; damp; sand is medium to fine; moderate to light, orange colored mottling decreasing with depth; gravel and cobbles are rounded; occasional boulders and minor coal present. **[Glacial Drift]**	
5.0					
	GB			Sample at 6': 31% Fines	
				6.5	245.5
		SM		(SM) SILTY SAND, some gravel; ~20-30-% fines; grayish brown; dense; moderate cohesion; non-plastic; damp to dry; sand is medium to fine; gravel is rounded and mostly fine; cemented and blocky at TD. **[Glacial Till]**	
				7.0	245.0

Bottom of test pit at 7.0 feet.

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TEST PIT NUMBER TP2

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 237' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES Seepage and caving at approximately 3' bgs.
4hrs AFTER EXCAVATION 3.00 ft / Elev 234.00 ft

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	236.5
0.5					
	GB	SP-SM		(SP-SM) POORLY GRADED SAND WITH SILT, some gravel and cobbles; ~10-20% fines; tan to gray; medium dense to dense; non cohesive; non-plastic; moist to saturated at depth; sand is medium to fine; moderate orange mottling throughout, decreasing around 3'; heavy orange oxidation staining ~2'; seepage and caving ~3'; refusal on rock. **[Glacial Outwash]**	
				Sample at 2': 11% Fines	
2.5					
4.0					233.0
				SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**	

Bottom of test pit at 4.0 feet.

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TEST PIT NUMBER TP3

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 229' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
			1.3		227.7
		SM		(SM) SILTY SAND, some gravel and cobbles, a little clay; ~20-30% fines; gray; medium dense; low cohesion; non-plastic; moist; sand is medium to fine; heavy orange mottling throughout; gravel and cobbles are rounded; occasional boulders present. **[Glacial Drift]**	
2.5			2.5		226.5
		SM		(SM) SILTY SAND; ~30-40% fines; grayish brown; medium dense to dense; moderate cohesion; low to non-plastic; damp; weathered in upper 0.5', cemented and blocky near TD; refusal on rock. **[Glacial Till]**	
			3.5		225.5

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 3.5 feet.

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TEST PIT NUMBER TP4

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 255' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; brown; soft; cohesive, non-plastic; moist; refusal on rock. **[Topsoil]**	
0.3					254.7

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 0.3 feet.

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TEST PIT NUMBER TP5

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 267' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; abundant root material present. **[Topsoil]**	
			0.5		266.5
		SM		(SM) SILTY SAND, some gravel and cobbles; ~25-35% fines; gray to tan; medium dense to dense; low to moderate cohesion; non-plastic; damp; sand is medium to fine; light, orange colored mottling evenly distributed throughout; gravel and cobbles are rounded; refusal on rock. **[Glacial Drift]**	
2.5			2.5		264.5

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 2.5 feet.

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TEST PIT NUMBER TP6

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 266' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; damp to dry; root material present. **[Topsoil]**	
0.5					265.5
	GB	SM		(SM) SILTY SAND, some gravel and cobbles; ~20-30% fines; tan to yellowish brown; medium dense to dense; low cohesion; non-plastic; damp; sand is fine; gravel and cobbles are angular; refusal on rock. **[Highly Reworked Rock]**	
2.5					263.5

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 2.5 feet.

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TEST PIT NUMBER TP7

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 253' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; damp; root material present. **[Topsoil]**	252.5
0.5					
		SM		(SM) SILTY SAND, some gravel and cobbles; ~25-35% fines; gray to tan; medium dense to dense; low cohesion; non-plastic; damp; sand is medium to fine; orange mottling and oxidation staining throughout, concentrated ~1.5'-2'; gravel and cobbles are rounded; becomes cemented and blocky before TD; refusal on rock. **[Highly Weathered Glacial Till]**	251.0
2.0					

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 2.0 feet.

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TEST PIT NUMBER TP8

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 258' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
	GB	OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; damp; root material present. **[Topsoil]**	0.8 / 257.2
		SM		(SM) SILTY SAND, some gravel and cobbles, a little clay; ~40-50% fines; brown to grayish brown; medium dense; low to moderate cohesion; non-plastic; moist; sand is medium to fine; light, orange colored mottling throughout; gravel and cobbles are rounded; occasional boulders present. **[Glacial Drift]**	2.0 / 256.0
2.5		SC		(SC) CLAYEY SAND, some gravel, cobbles, and silt; ~20-30% fines; brown to gray; medium dense; low to moderate cohesion; medium plasticity; moist; heavy, orange colored redox mottling from ~2'-5'; increased gravel content from ~4'-6'; occasional boulders present. **[Glacial Drift]**	
5.0				Sample at 4': 20% Fines; Atterberg Limits: LL = 51, PL = 25, PI = 26	6.5 / 251.5
7.5		SM		(SM) SILTY SAND, some fine gravel; ~30-50% fines; grayish brown; dense; low to moderate cohesion; non-plastic; damp; sand is medium to fine; blocky and cemented; refusal in very dense weathered rock. **[Glacial Till]**	8.0 / 250.0

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 8.0 feet.

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TEST PIT NUMBER TP9

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 272' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	271.5
		ML		(ML) SANDY SILT; ~60-70% fines; grayish brown; stiff; cohesive; low to non-plastic; damp; light, orange colored mottling throughout; occasional boulders present; roots stop at ~2.3'. **[Glacial Drift]**	
2.5					269.5
	GB	SM		(SM) SILTY SAND, some gravel and cobbles; ~20-30% fines; orange brown; dense; low to moderate cohesion; non-plastic; damp; sand is medium to fine; transitions to weathering rind before refusal on rock. **[Glacial Drift]** Sample at 4': 22% Fines	267.5
				SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]** Bottom of test pit at 4.5 feet.	

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TEST PIT NUMBER TP10

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 279' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
0.4					278.6
		SM		(SM) SILTY SAND, some gravel; ~20-30% fines; tan to yellowish brown; medium dense to dense; low cohesion; non-plastic; damp to dry; sand is fine; gravel is angular; refusal on rock. **[Highly Reworked Rock]**	
2.5					
	GB			Sample at 3': 21% Fines	
3.5					275.5

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 3.5 feet.

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TEST PIT NUMBER TP11

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 290' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	289.6
0.4					
		SM		(SM) SILTY SAND, a little clay; ~30-40% fines; light brown to grayish; medium dense; low to moderate cohesion; non-plastic; damp; sand is medium to fine; moderate, orange colored mottling to ~1.5'-3'; refusal on rock. **[Glacial Drift]**	
2.5					
3.0					287.0

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 3.0 feet.

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TEST PIT NUMBER TP12

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 292' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
			0.4		291.6
		SM		(SM) SILTY SAND, a little clay; ~30-40% fines; light brown to grayish; medium dense; low to moderate cohesion; non-plastic; damp; sand is medium to fine; moderate, orange colored mottling from ~2'-3.3'; transitions to weathering rind before refusal on rock. **[Glacial Drift]**	
2.5					
	GB			Sample at 3': 28% Fines	
			3.5		288.5

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 3.5 feet.

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TEST PIT NUMBER TP13

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 238' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT, some gravel and cobbles; dark brown to orange brown; soft to medium stiff; cohesive, non-plastic; moist; root material present, disturbed - some buried garbage. **[Topsoil]**	
2.5					
				3.0	235.0
	GB	GP		(GP) POORLY GRADED GRAVEL WITH SAND, some cobbles; <10% fines; brown; medium dense; non-cohesive; non-plastic; moist. **[Glacial Outwash]**	
				Sample at 4': 2% Fines	
				4.5	233.5
5.0				(SM) SILTY SAND WITH GRAVEL; ~30-40% fines; light gray to gray; medium dense to dense ~6'; low to moderate cohesion; low to non-plastic; damp; gravel is fine. [Glacial Drift]	
	GB	SM		Sample at 6': 28% Fines	
				6.5	231.5
		SM		(SM) SILTY SAND, some gravel; ~20-30% fines; light gray to gray; dense; low to moderate cohesion; low to non-plastic; damp to dry; gravel is fine; cemented and blocky; refusal in hardpan till. **[Glacial Till]**	
				7.0	231.0

Bottom of test pit at 7.0 feet.

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TEST PIT NUMBER TP14

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 228' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
	GB	OL		(OL) ORGANIC SILT; brown to orange brown; soft; cohesive, non-plastic; damp; abundant root material present. **[Topsoil]**	1.5 / 226.5
2.5		SM		(SM) SILTY SAND, some cobbles; ~20-30% fines, variable; light grayish brown; medium dense; non-cohesive; non-plastic; damp; large boulders present. **[Glacial Outwash]**	4.0 / 224.0
5.0		SM		(SM) SILTY SAND, some gravel; ~40-50% fines; light gray to gray; medium dense to dense; moderate cohesion; low to non-plastic; damp; gravel is fine. **[Glacial Drift]**	6.0 / 222.0
		SM		(SM) SILTY SAND, some gravel; ~30-40% fines; light gray to gray; dense; low to moderate cohesion; low to non-plastic; damp to dry; gravel is fine; cemented and blocky; refusal in hardpan till. **[Glacial Till]**	7.0 / 221.0

Bottom of test pit at 7.0 feet.

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TEST PIT NUMBER TP15

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 6/30/20 **COMPLETED** 6/30/20
GROUND ELEVATION 223' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown to orange brown; soft; cohesive, non-plastic; damp; root material present. **[Topsoil]**	
2.5					
3.0					220.0
		SW		(SW) WELL GRADED SAND WITH GRAVEL, some cobbles; <10% fines; grayish brown; medium dense; non-cohesive; non-plastic; moist; gravel and cobbles are well-rounded; refusal on large boulder. **[Glacial Outwash]**	
5.0					
6.0					217.0

Bottom of test pit at 6.0 feet.

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TEST PIT NUMBER TP16

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 254' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT, some cobbles; dark brown to dark reddish orange brown; soft; cohesive, non-plastic; moist; root material present, cobbles are rounded to well-rounded. **[Topsoil]**	
1.5					252.5
		GP-GM		(GP-GM) POORLY-GRADED GRAVEL WITH SILT AND SAND, some cobbles; <10% fines, variable; brown to orange brown; medium dense; non-cohesive; non-plastic; moist; some boulders present. **[Glacial Outwash]**	
2.5					
	GB			Sample at 3': 5% Fines	
3.5					250.5
		SC-SM		(SC-SM) SILTY, CLAYEY SAND, some medium to fine gravel; ~30-40% fines; light gray to gray; dense to very dense; low to moderate cohesion; low plasticity; damp; cemented and blocky; transitions to weathering rind before refusal on rock. **[Glacial Till]**	
4.5					249.5
	GB			Sample at 4.5': 39% Fines; Atterberg Limits: LL=21, PL=16, PI=5	
				SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**	

Bottom of test pit at 4.5 feet.

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TEST PIT NUMBER TP17

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 258' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT, some cobbles; dark brown to orange brown; soft; cohesive, non-plastic; moist; root material present, cobbles are rounded to well-rounded. **[Topsoil]**	
			1.8		256.2
2.5		SP		(SP) POORLY-GRADED SAND WITH SILT; ~10-20% fines, variable; tan to gray; medium dense; non-cohesive; non-plastic; damp; some boulders present. **[Glacial Outwash]**	
			3.0		255.0
		SM		(SM) SILTY SAND, some medium to fine gravel; ~30-40% fines; light gray to gray; dense to very dense; low to moderate cohesion; non-plastic; damp; weathered in upper 1', cemented and blocky near TD; transitions to weathering rind before refusal on rock. **[Glacial Till]**	
			4.5		253.5

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 4.5 feet.

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TEST PIT NUMBER TP18

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 265' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
	GB	OL		(OL) ORGANIC SILT, some cobbles; dark brown to orange brown; soft; cohesive, non-plastic; moist; root material present; cobbles are rounded to well-rounded. **[Topsoil]**	263.0
2.0					
2.5		SM		(SM) SILTY SAND, some gravel and cobbles; ~20-40% fines, variable; brown to orange brown; medium dense; low to moderate-cohesion; non-plastic; damp to moist; gravel clasts are rounded. **[Glacial Drift]**	261.0
4.0					
5.0		SP-SM		(SP-SM) POORLY-GRADED SAND WITH SILT; ~10% fines; tan to gray; dense to very dense; non-cohesive; non-plastic; dry; transitions to weathering rind before refusal on intact rock. **[Highly Reworked Rock]**	260.0
5.0				SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**	

Bottom of test pit at 5.0 feet.

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TEST PIT NUMBER TP19

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 278' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION		
0.0						
		OL		(OL) ORGANIC SILT; dark brown to orange brown; soft; cohesive, non-plastic; damp to moist; abundant root material present. **[Topsoil]**	2.0	276.0
2.5		SM		(SM) SILTY SAND, some gravel and cobbles; ~20-40% fines, variable; brown to orange brown; medium dense; low to moderate-cohesion; non-plastic; moist; some light gray silt lensing with orange redox mottling; gravel clasts are rounded. **[Glacial Drift]**	4.0	274.0
5.0		SP-SM		(SP-SM) POORLY-GRADED SAND WITH SILT; ~10% fines; tan to gray; dense to very dense; non-cohesive; non-plastic; dry; transitions to weathering rind before refusal on intact rock. **[Highly Reworked Rock]**	5.0	273.0

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 5.0 feet.

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TEST PIT NUMBER TP20

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 288' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown to orange brown; soft; cohesive, non-plastic; moist; abundant root material present. **[Topsoil]**	
			1.5		286.5
		SM		(SM) SILTY SAND, little gravel and cobbles; ~30-40% fines; light grayish brown; medium dense; low cohesion; non-plastic; damp; some orange oxidation around ~2.5'-3.5'. **[Glacial Drift]**	
2.5					
			3.5		284.5
		ML		(ML) SANDY SILT, some fine gravel; ~40-60% fines, variable; light gray to gray; stiff; cohesive; low to non-plastic; moist; orange colored mottling throughout. **[Glacial Drift]**	
5.0			5.0		283.0
		SM		(SM) SILTY SAND, some medium to fine gravel; ~30-40% fines; light gray to gray; dense to very dense; low to moderate cohesion; non-plastic; damp; weathered in upper ~1', cemented and blocky; refusal in hardpan till. **[Glacial Till]**	
			7.0		281.0

Bottom of test pit at 7.0 feet.

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TEST PIT NUMBER TP21

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 282' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown to reddish brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
1.0					281.0
		SM		(SM) SILTY SAND, some gravel and cobbles; ~20-30% fines; tan to yellowish brown; medium dense to very dense; low cohesion; non-plastic; damp; sand is medium to fine; gravel and cobbles are angular (weathered sandstone); refusal on rock. **[Highly Reworked Rock]**	
2.5					
3.0					279.0

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 3.0 feet.

Element Solutions
909 Squalicum Way, Suite 111
Bellingham, WA 98225
Telephone: (360) 671-9172
Fax: (360) 671-4685

TEST PIT NUMBER TP22

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 290' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown to brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
				1.5	288.5
		SM		(SM) SILTY SAND, little gravel and cobbles; ~30-40% fines; light grayish brown; medium dense; low to moderate cohesion; non-plastic; damp to moist; clasts are rounded; occasional boulders present. **[Glacial Drift]**	
2.5					
				3.5	286.5
		SM		(SM) SILTY SAND, some fine gravel; ~20-30% fines; gray; dense to very dense; low to moderate cohesion; non-plastic; damp; some light, orange colored redox mottling around interface with overlying unit; cemented and blocky; transitions to weathering rind before refusal on rock. **[Glacial Till]**	
5.0				5.0	285.0
				SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]** Bottom of test pit at 5.0 feet.	

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TEST PIT NUMBER TP23

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 305' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
				1.0	304.0
		SM		(SM) SILTY SAND, little gravel and cobbles; ~30-40% fines; light grayish brown; medium dense; low to moderate cohesion; non-plastic; damp to moist; some light, orange colored redox mottling ~2'-3'; clasts are rounded; occasional boulders present. **[Glacial Drift]**	
2.5					
				3.0	302.0
		SM		(SM) SILTY SAND, some fine gravel; ~20-30% fines; gray; dense to very dense; low to moderate cohesion; non-plastic; damp; some light, orange colored redox mottling around interface with overlying unit; cemented and blocky; transitions to weathering rind before refusal on rock. **[Glacial Till]**	
				4.5	300.5
				SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**	

Bottom of test pit at 4.5 feet.

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TEST PIT NUMBER TP24

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 227' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
	GB	OL		(OL) ORGANIC SILT, some cobbles and gravel; dark brown to reddish brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
1.5					225.5
		SP-SM		(SP-SM) POORLY-GRADED SAND WITH SILT AND GRAVEL, some large cobbles; ~5-10% fines; grayish brown; medium dense; non-cohesive; non-plastic; moist; some boulders present. **[Glacial Drift]**	
2.5					
				Sample at 4': 8% Fines	
4.2					222.8
		SM		(SM) SILTY SAND, some medium to fine gravel; ~20-30% fines; gray; dense to very dense; low cohesion; non-plastic; damp; cemented and blocky; refusal on rock. **[Glacial Till]**	
5.0					
5.5					221.5
				SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**	
				Bottom of test pit at 5.5 feet.	

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TEST PIT NUMBER TP25

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 332' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
0.8					331.2
		SP		(SP) POORLY-GRADED SAND; <5% fines; yellowish brown; medium dense to very dense; non-cohesive; non-plastic; damp to moist; transitions to weathering rind before refusal on rock. **[Eluvium]**	
2.5	GB			Sample at 2.5': 2% Fines	
4.0					328.0

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 4.0 feet.

Element Solutions
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Telephone: (360) 671-9172
Fax: (360) 671-4685

TEST PIT NUMBER TP26

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION Viewcrest Road, Bellingham, WA
DATE STARTED 7/1/20 **COMPLETED** 7/1/20
GROUND ELEVATION 340' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Ryan Bradley
GROUND WATER LEVELS:
EXCAVATION METHOD Yanmar compact excavator
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES No groundwater or free water seepage observed.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
0.5					339.5
		SP		(SP) POORLY-GRADED SAND; <5% fines; yellowish brown; loose to medium dense becoming dense to very dense ~3.5'; non-cohesive; non-plastic; damp to moist; transitions to weathering rind before refusal on rock. **[Eluvium]**	
2.5					
4.0					336.0

SANDSTONE BEDROCK; tan; very dense. **[Chuckanut Formation (Padden Member)]**
Bottom of test pit at 4.0 feet.

GEOTEST

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www.geotest-inc.com

July 16, 2020

Job Number: 20-0587
Job Name: Jones-Edgemoor Estates
Client: Element Solutions
Address: Whatcom County, WA

As requested, GeoTest Services, Inc. performed materials testing services at our Bellingham, WA laboratory for the project noted above. The testing was performed in accordance with the applicable ASTM/AASHTO test methods. Please see the attached laboratory reports and summary of the test results listed in the chart below:

Sample Number: 1166

Test(s) Performed:	Pass / Fail	Comments:
Sieve - ASTM C136/C117	N/A	
Hydrometer - ASTM D422	N/A	

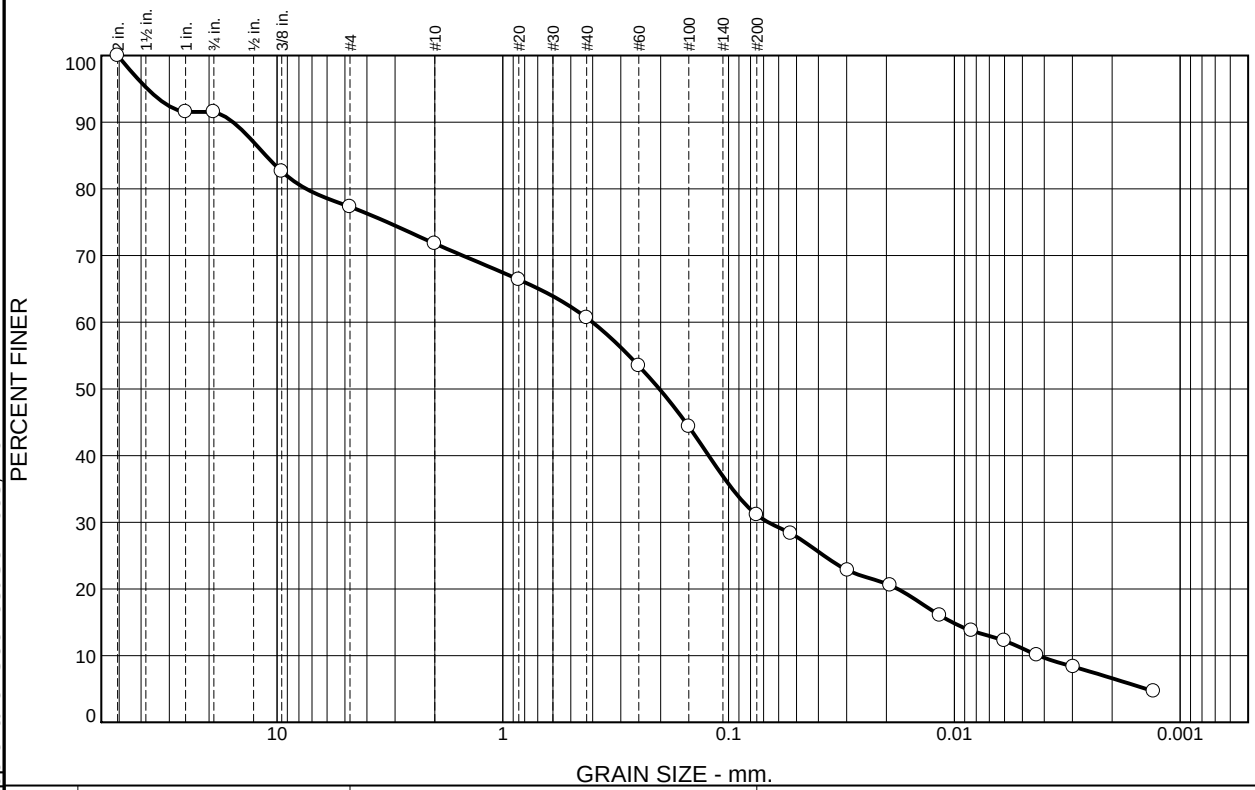
We appreciate the opportunity to be of service to you on this project. If you have any questions regarding the results of the test(s) performed, methods used, or require any other assistance, please don't hesitate to contact the undersigned.

Sincerely,

David Bufalini

David Bufalini, Supervising Lab Technician
daveb@geotest-inc.com
360.410.8170 (c)

Sieve Analysis w/Hydrometer Test Report - D422/D1140



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	8	15	5	11	30	20	11

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1"	92		
3/4"	92		
3/8"	83		
#4	77		
#10	72		
#20	66		
#40	61		
#60	53		
#100	44		
#200	31		
#270	28		
0.0297 mm.	23		
0.0192 mm.	21		
0.0116 mm.	16		
0.0084 mm.	14		
0.0060 mm.	12		
0.0043 mm.	10		
0.0030 mm.	8.3		
0.0013 mm.	4.7		

* (no specification provided)

Soil Description

Native Material
silty sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 15.6391 D_{85}= 11.1904 D_{60}= 0.4007
D_{50}= 0.2026 D_{30}= 0.0668 D_{15}= 0.0102
D_{10}= 0.0042 C_u= 94.53 C_c= 2.63

Classification

USCS= SM AASHTO=

Remarks

No specification provided.

Location: Native Material - Sampled by Client from TP-1 @ 6'
Sample Number: 1166 **Date:** 7-16-20

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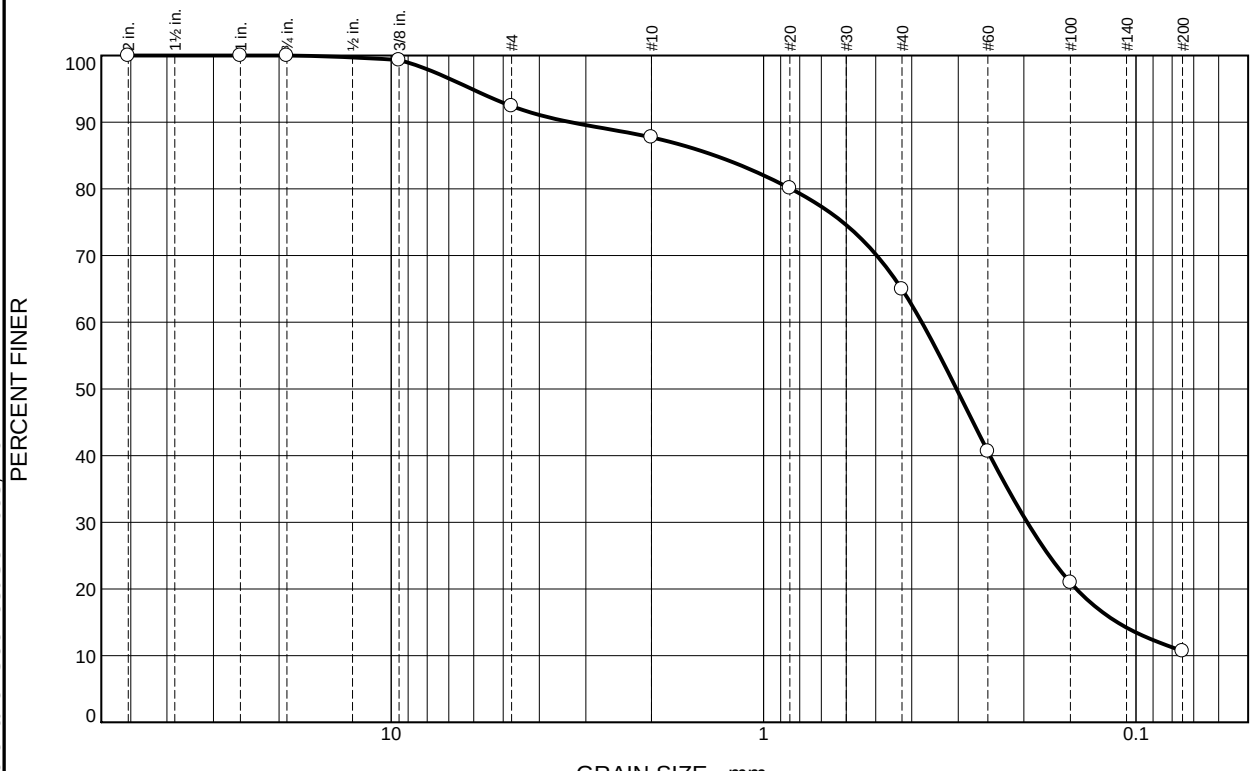
Client: Element Solutions
Project: Jones-Edgemoor Estates
Whatcom County, WA
Project No: 20-0587 **Figure** SA001

Tested By: DK **Checked By:** DB

Sieve Analysis Test Report - ASTM C136/C117



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt
0	0	8	4	23	54	11

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1"	100		
3/4"	100		
3/8"	99		
#4	92		
#10	88		
#20	80		
#40	65		
#60	41		
#100	21		
#200	11		

* (no specification provided)

Soil Description

Native Material
poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.2938 D_{85}= 1.3497 D_{60}= 0.3761
D_{50}= 0.3034 D_{30}= 0.1959 D_{15}= 0.1122
D_{10}= C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

No specification provided.

Location: Native Material - Sampled by Client from TP-2 @ 2'
Sample Number: 1167 **Date:** 7-16-20

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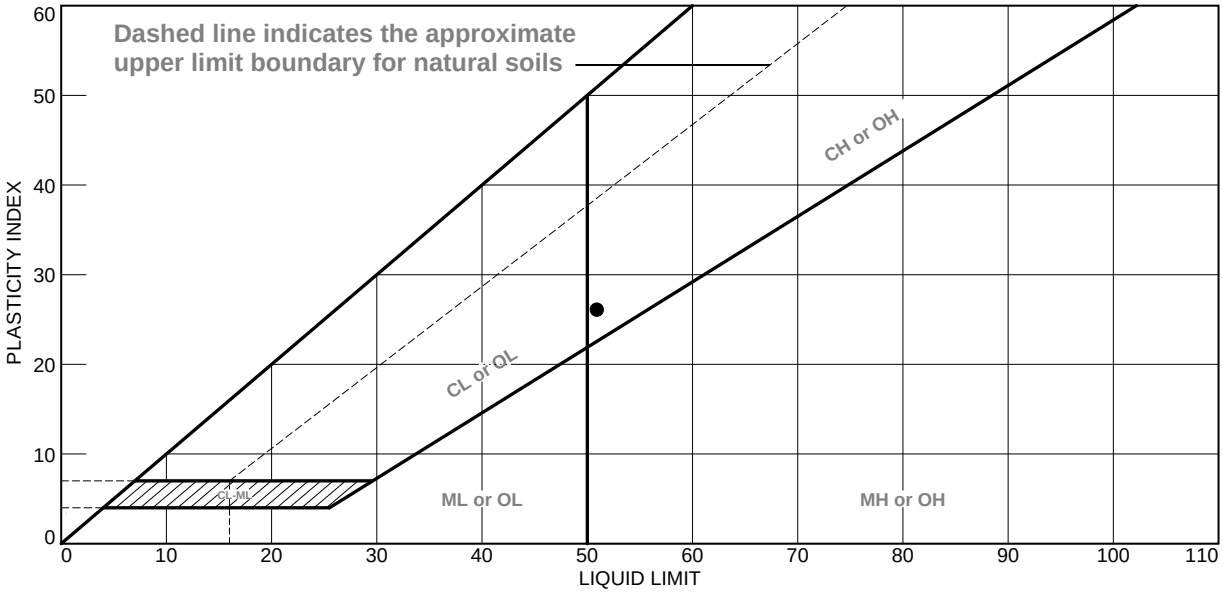
Client: Element Solutions
Project: Jones-Edgemoor Estates
Whatcom County, WA
Project No: 20-0587 **Figure** SA002

Tested By: DK **Checked By:** DB

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Native Material poorly graded sand with silt	51	25	26		20	

Project No. 20-0587 **Client:** Element Solutions

Project: Jones-Edgemoor Estates

Whatcom County, WA

Location: Native Material - Sampled by Client from TP-8 @ 4'

Sample Number: 1168

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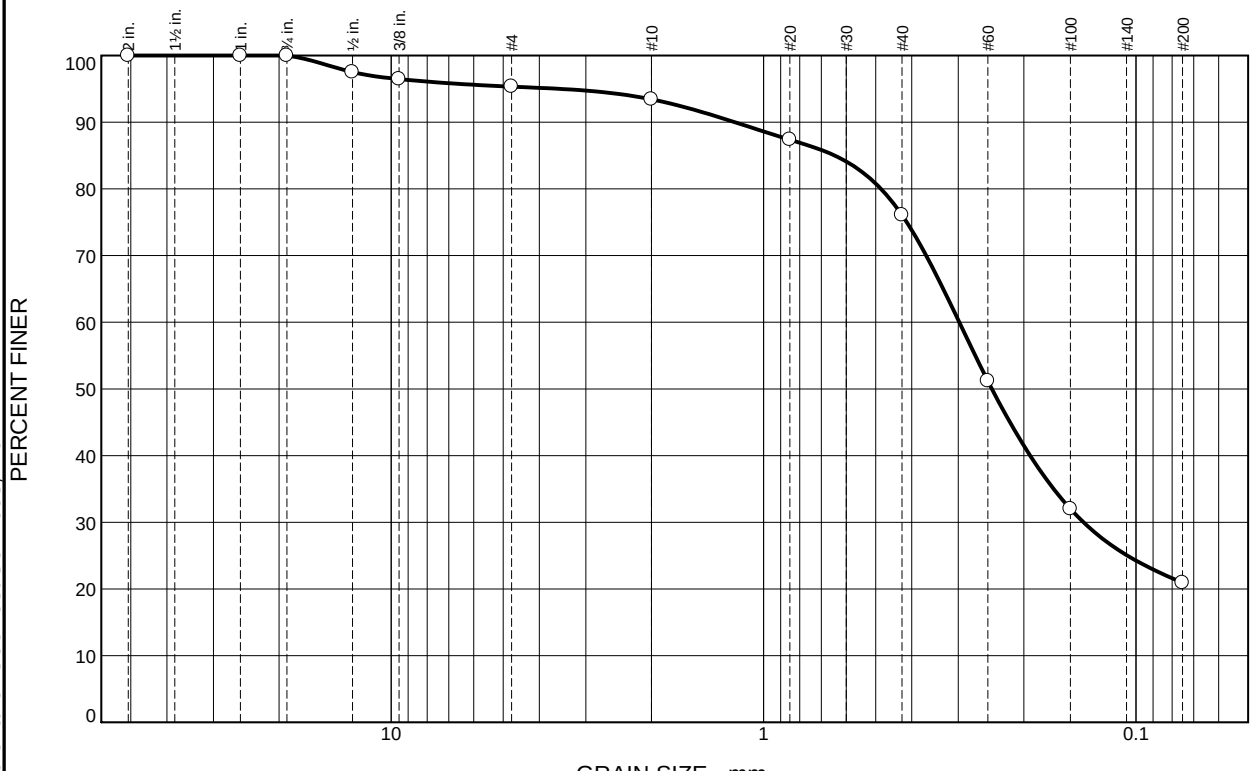
Remarks:

●Percent Passing #200: 20.2%

Figure SA003

Tested By: MY **Checked By:** DB

Sieve Analysis Test Report - ASTM C136/C117



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt
0	0	5	2	17	55	21

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1"	100		
3/4"	100		
1/2"	98		
3/8"	96		
#4	95		
#10	93		
#20	87		
#40	76		
#60	51		
#100	32		
#200	21		

* (no specification provided)

Soil Description

Native Material
silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.1937 D_{85}= 0.6440 D_{60}= 0.2983
D_{50}= 0.2438 D_{30}= 0.1384 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

No specification provided.

Location: Native Material - Sampled by Client from TP-10 @ 3'
Sample Number: 1170 **Date:** 7-16-20

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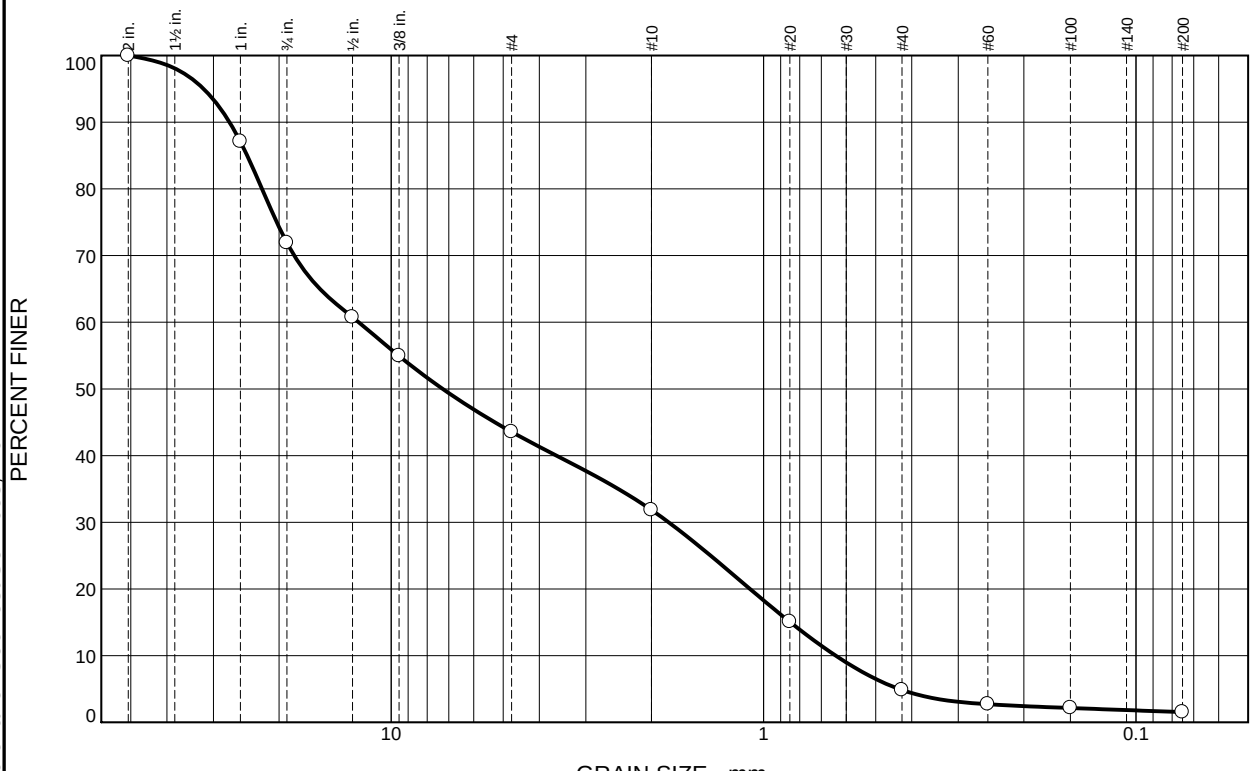
Client: Element Solutions
Project: Jones-Edgemoor Estates
Whatcom County, WA
Project No: 20-0587 **Figure** SA004

Tested By: DK **Checked By:** DB

Sieve Analysis Test Report - ASTM C136/C117



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt
0	28	28	12	27	3	2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1"	87		
3/4"	72		
1/2"	61		
3/8"	55		
#4	44		
#10	32		
#20	15		
#40	5		
#60	3		
#100	2		
#200	1.5		

* (no specification provided)

Soil Description

Native Material
poorly graded gravel with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 27.1600 D_{85}= 24.3550 D_{60}= 12.2231
D_{50}= 7.2691 D_{30}= 1.7986 D_{15}= 0.8474
D_{10}= 0.6405 C_u= 19.08 C_c= 0.41

Classification

USCS= GP AASHTO=

Remarks

No specification provided.

Location: Native Material - Sampled by Client from TP-13 @ 4'
Sample Number: 1172 **Date:** 7-16-20

GEOTEST
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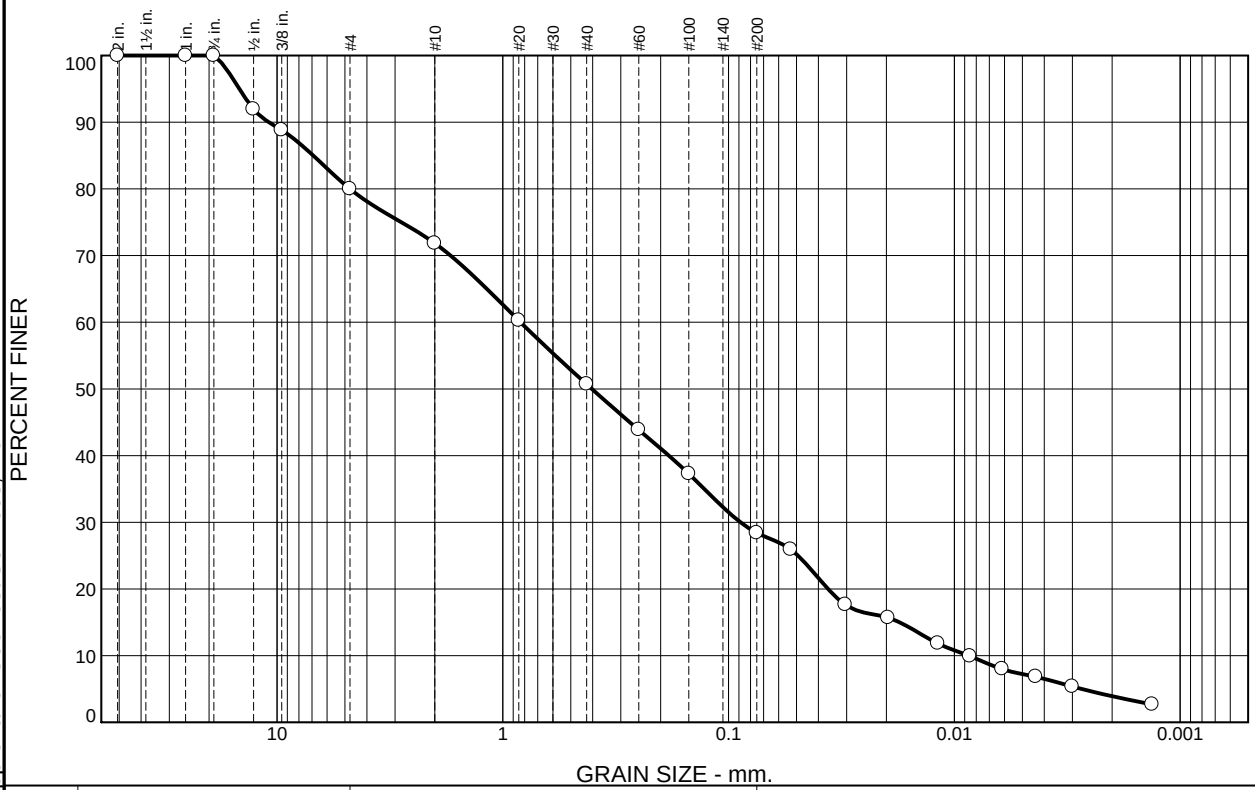
Client: Element Solutions
Project: Jones-Edgemoor Estates
Whatcom County, WA
Project No: 20-0587 **Figure** SA005

Tested By: DK **Checked By:** DB

Sieve Analysis w/Hydrometer Test Report - D422/D1140



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	20	8	21	23	21	7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1"	100		
3/4"	100		
1/2"	92		
3/8"	89		
#4	80		
#10	72		
#20	60		
#40	51		
#60	44		
#100	37		
#200	28		
#270	26		
0.0304 mm.	18		
0.0196 mm.	16		
0.0118 mm.	12		
0.0085 mm.	9.9		
0.0061 mm.	8.0		
0.0044 mm.	6.8		
0.0030 mm.	5.4		
0.0013 mm.	2.7		

* (no specification provided)

Soil Description

Native Material
silty sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 10.8708 D_{85}= 6.9089 D_{60}= 0.8348
D_{50}= 0.4032 D_{30}= 0.0886 D_{15}= 0.0175
D_{10}= 0.0087 C_u= 96.49 C_c= 1.09

Classification

USCS= SM AASHTO=

Remarks

No specification provided.

Location: Native Material - Sampled by Client from TP-13 @ 6'
Sample Number: 1173 **Date:** 7-16-20

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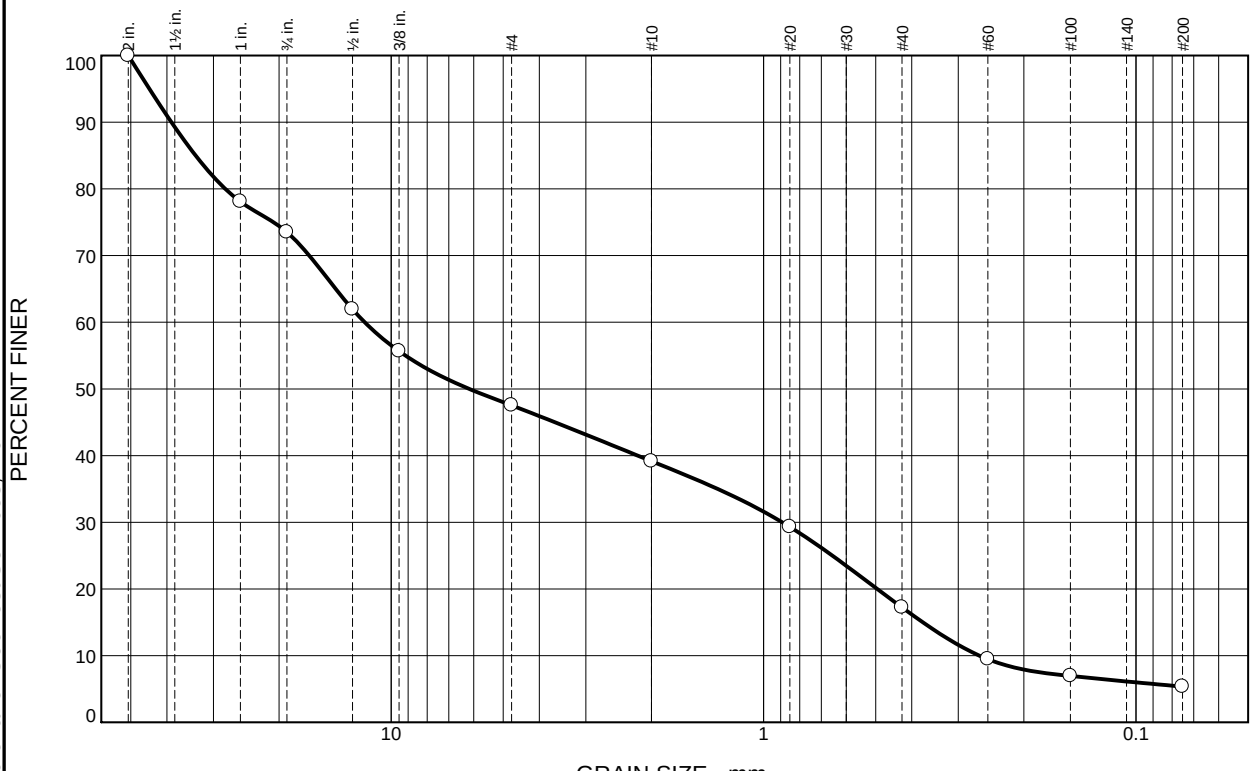
Client: Element Solutions
Project: Jones-Edgemoor Estates
Whatcom County, WA
Project No: 20-0587 **Figure** SA006

Tested By: DK **Checked By:** DB

Sieve Analysis Test Report - ASTM C136/C117



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt
0	26	26	9	22	12	5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1"	78		
3/4"	74		
1/2"	62		
3/8"	56		
#4	48		
#10	39		
#20	29		
#40	17		
#60	9		
#100	7		
#200	5.3		

* (no specification provided)

Soil Description

Native Material
poorly graded gravel with silt and sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 38.9059 D_{85}= 33.4966 D_{60}= 11.7814
D_{50}= 6.1688 D_{30}= 0.8908 D_{15}= 0.3738
D_{10}= 0.2635 C_u= 44.71 C_c= 0.26

Classification

USCS= GP-GM AASHTO=

Remarks

No specification provided.

Location: Native Material - Sampled by Client from TP-16 @ 3'
Sample Number: 1174 **Date:** 7-16-20

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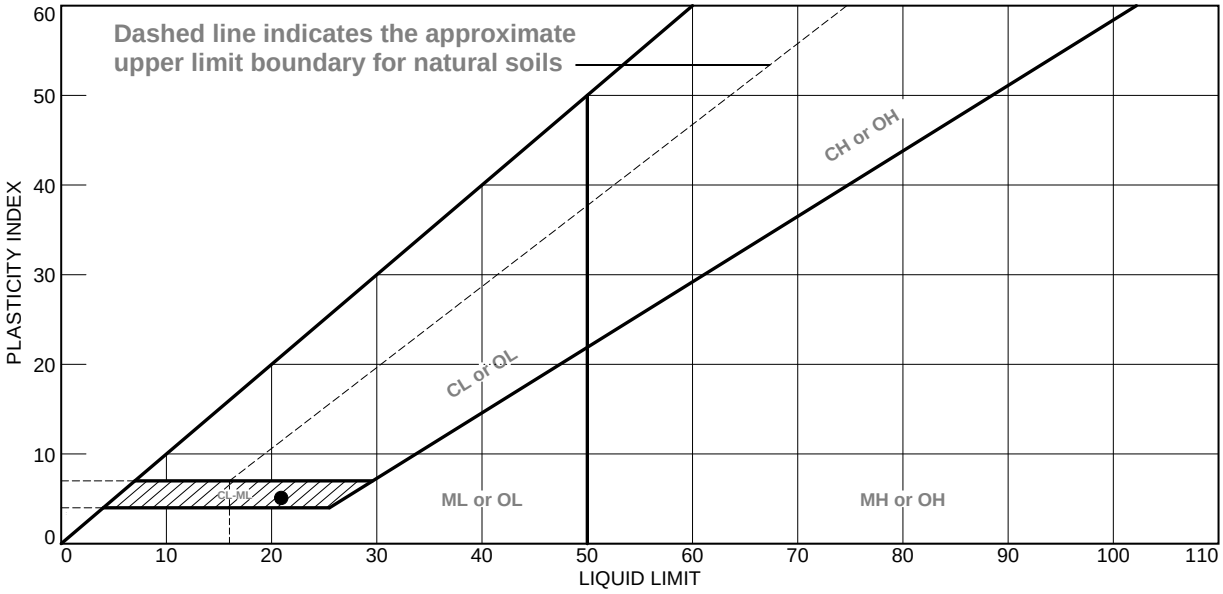
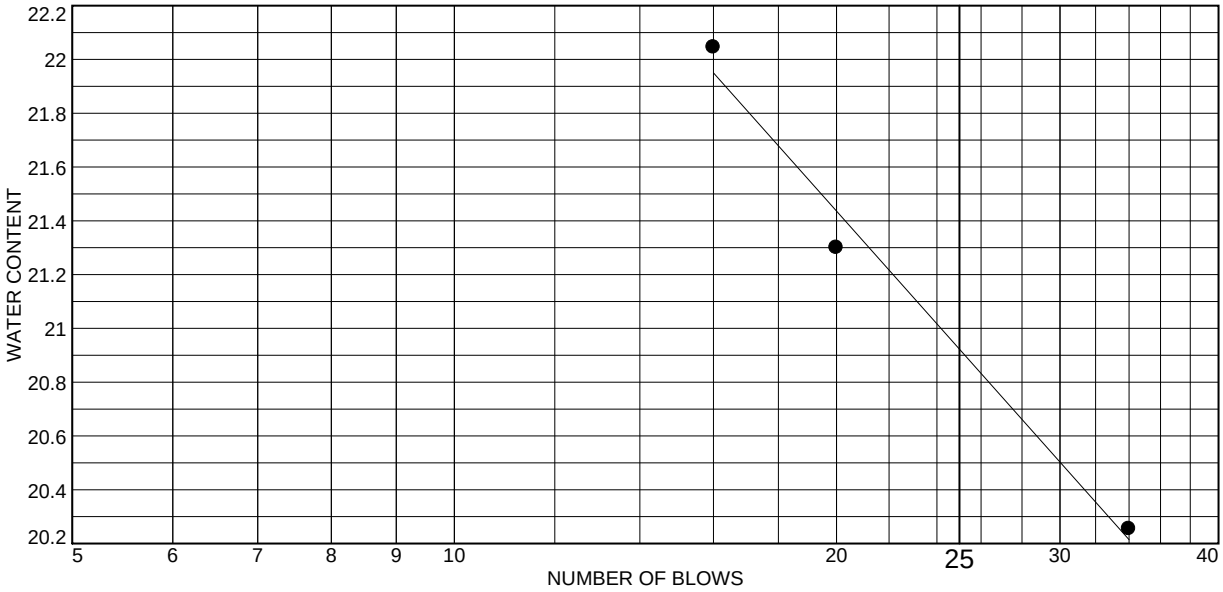
Client: Element Solutions
Project: Jones-Edgemoor Estates
Whatcom County, WA
Project No: 20-0587 **Figure** SA007

Tested By: DK **Checked By:** DB

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Native Material poorly graded sand with silt	21	16	5	71	39	SC-SM

Project No. 20-0587 **Client:** Element Solutions

Project: Jones-Edgemoor Estates

Whatcom County, WA

Location: Native Material - Sampled by Client from TP-16 @ 4.5'

Sample Number: 1175

Remarks:

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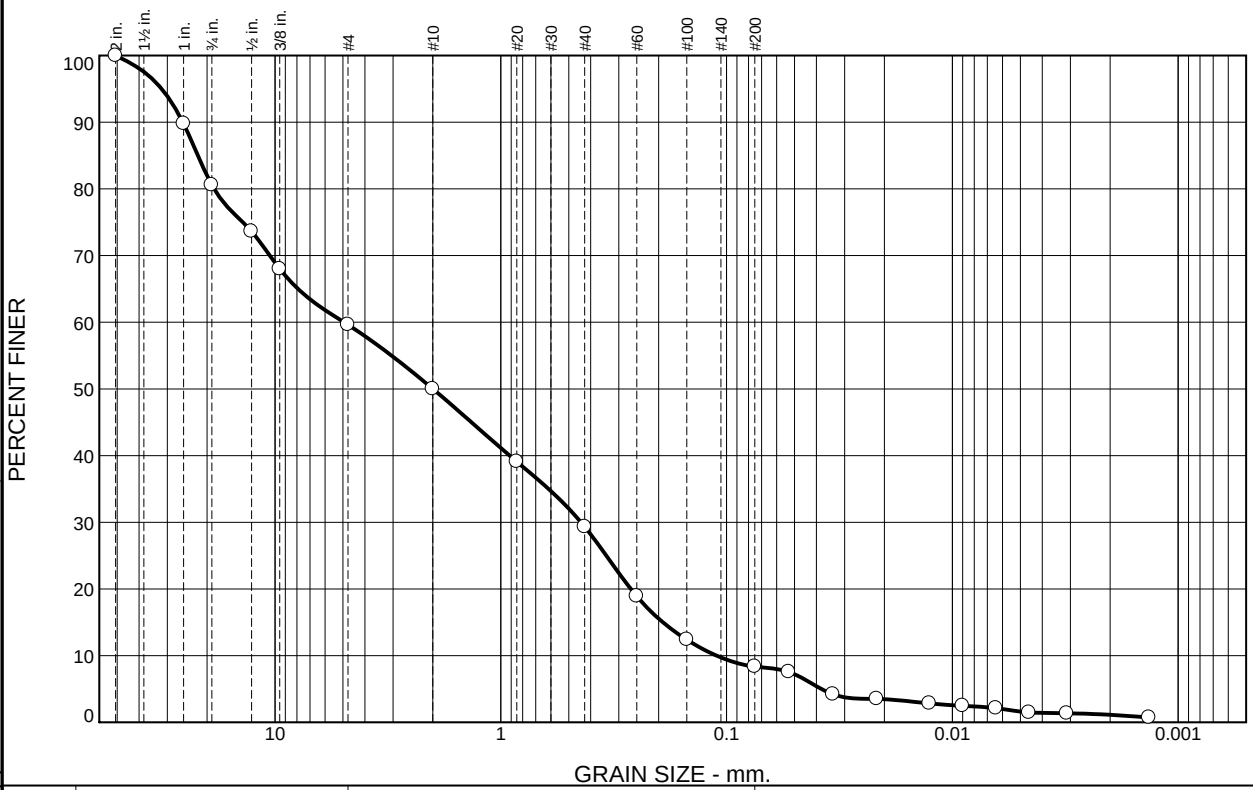
Figure SA008

Tested By: MY **Checked By:** DB

Sieve Analysis w/Hydrometer Test Report - D422/D1140



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	19	21	10	21	21	6	2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1"	90		
3/4"	81		
1/2"	74		
3/8"	68		
#4	60		
#10	50		
#20	39		
#40	29		
#60	19		
#100	12		
#200	8.3		
#270	7.6		
0.0338 mm.	4.2		
0.0216 mm.	3.5		
0.0126 mm.	2.8		
0.0090 mm.	2.5		
0.0064 mm.	2.1		
0.0046 mm.	1.5		
0.0031 mm.	1.3		
0.0013 mm.	0.7		

* (no specification provided)

Soil Description

Native Material
poorly graded sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 25.5937 D_{85}= 21.9658 D_{60}= 4.9435
D_{50}= 1.9989 D_{30}= 0.4413 D_{15}= 0.1915
D_{10}= 0.1109 C_u= 44.59 C_c= 0.36

Classification

USCS= SP-SM AASHTO=

Remarks

No specification provided.

Location: Native Material - Sampled by Client from TP-24 @ 4'
Sample Number: 1176 **Date:** 7-16-20

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Client: Element Solutions
Project: Jones-Edgemoor Estates
Whatcom County, WA
Project No: 20-0587 **Figure** SA009

Tested By: DK **Checked By:** DB

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PERCENT PASSING #200

PROJECT:	**Jones-Edgemoor Estates**	JOB #:	**20-0587**
ADDRESS:	**Whatcom County, WA**	REPORT #:	**MR001**
PERMIT #:		DATE:	**7-16-2020**
CLIENT:	**Element Solutions**	PAGE #:	**1 of 1**
CONTRACTOR:	**N/A**	LAB #:	**1169, 1171, 1177**

Material Use: **Native Material**
Specification: **N/A**

Laboratory Test Data:

GeoTest Lab #	Source	Test Reference	% Passing #200
1169	TP-9 @ 4’	ASTM C117	22.1
1171	TP-12 @ 3’	ASTM C117	27.8
1177	TP-25 @ 2.5’	ASTM C117	2.34

Comments:

Copies: Element Solutions

David Bufalini

Reviewed by

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PAP-Accredited
NAPT

Element Solutions
1812 Cornwall Ave.
Bellingham, WA 98225

Report: 52022-1-1
Date: July 21, 2020
Project No: 2020094
Project Name: Jones Edgemoor Estate

Sample ID	Organic Matter	Cation Exchange Capacity
TP-1 @ 6.0’	1.77%	11.6 meq/100g
TP-13 @ 4.0’	1.50%	3.9 meq/100g
TP-24 @ 4.0’	1.44%	6.2 meq/100g
Method	**ASTM D2974**	**EPA 9081**

Exhibit A – Jones Edgemoor Estate - Test Pit Field Photos

Photo 1: TP1 Subsurface; Fill over Glacial Drift and Till

Photo 2: TP2; Excavated Boulder

Photo 3: TP5; Redox Staining in Shallow Soil

Photo 4: TP8 Location; Excavated Glacial Drift

Photo 5: TP9; Oxidized Soil Horizon

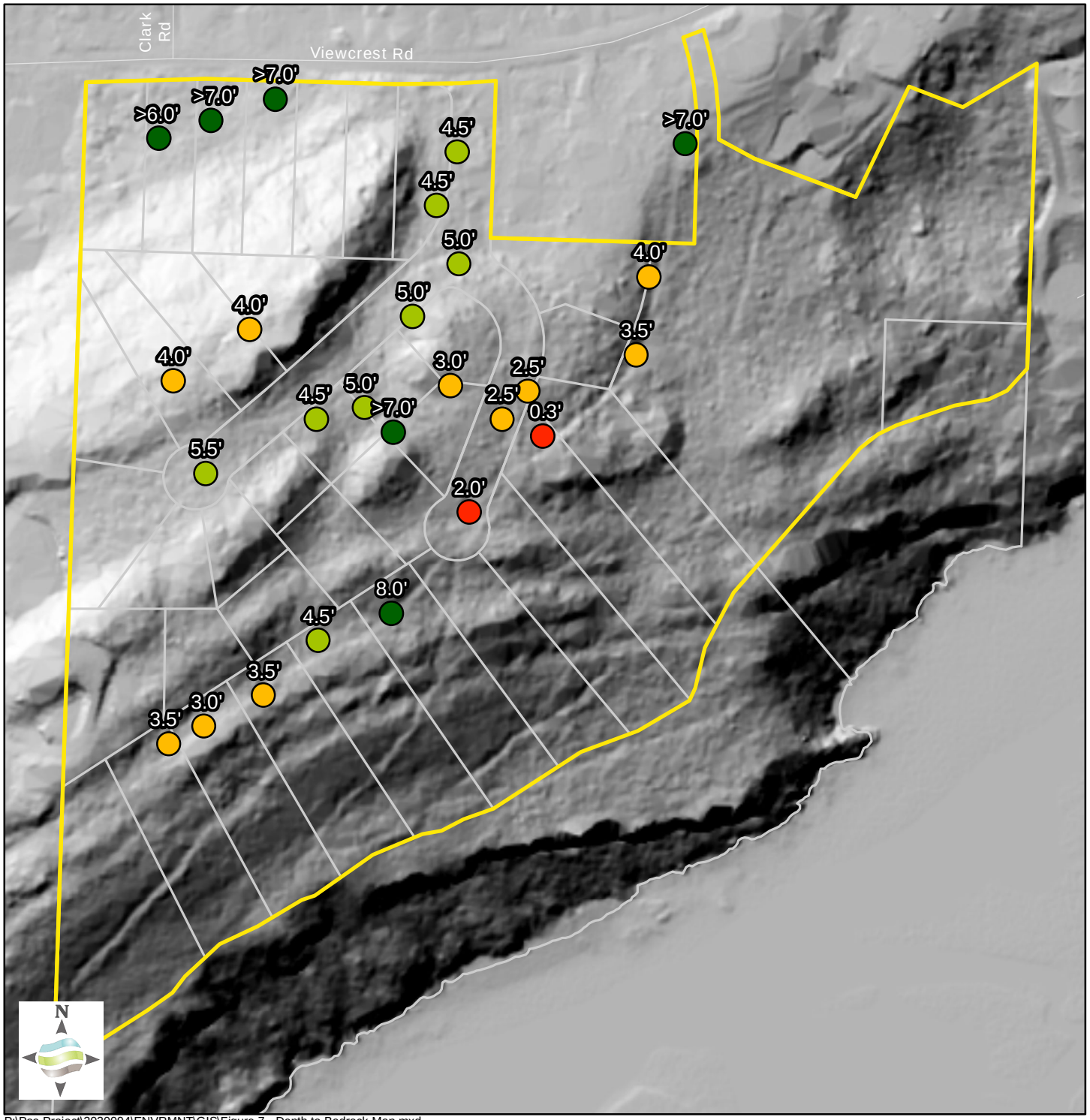
Photo 6: TP13; Thick Organic over Glacial Outwash and Dense Till

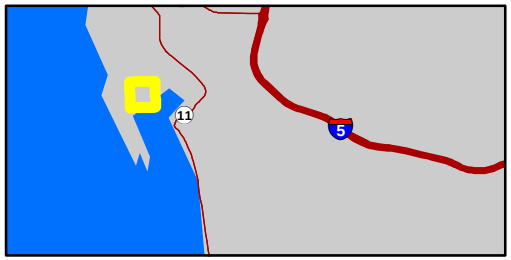
Photo 7: TP15; Thick Organic over Glacial Outwash

Photo 8: TP17; Dense Glacial Till at Base of Pit

Photo 9: TP23; Glacial Drift over Dense Till

Photo 10: TP25; Sandy Eluvium Over Dense Sandstone Bedrock





Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Lidar] COB 2013

Study Area
Lot Layout
Roads

Approx. Depth to Bedrock [ft]

- 0 - 2
- 2 - 4
- 4 - 6
- 6+ (Bedrock not Encountered)

Feet
0 250 500

1:2,400

1 inch = 200 feet

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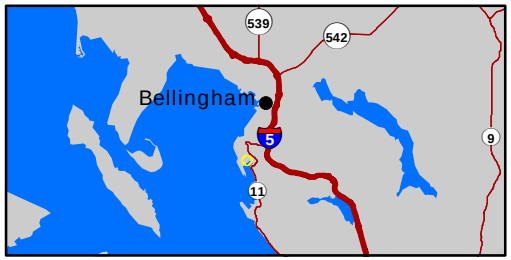
Figure 7

Jones - Edgemoor Estate
Approximate Depth to Bedrock Map

Date: 2/25/2022

P:\Pse Project\2020094\ENVRMNT\GIS\Figure 7 - Depth to Bedrock Map.mxd



Data Credits:
[Lidar] COB 2013
[Aerial] COB 2019
[Topo] Pacific Surveying & Engineering

Test Pit

Hand Auger

Proposed Sewer Line

Topo Lines

0 5 10 20 30 40 50
Feet

1:240

1 inch = 20 feet

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Figure 8

Jones - Edgemoor Estate
Sea Pines Rd Field Map

Date: 7/23/2021

P:\Pse Project\2020094\ENVRMNT\GIS\Figure 8 - Sea Pines Field Map.mxd

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Fax: (360) 671-4685

BORING NUMBER HA-1

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION 315 Sea Pines Road, Bellingham, WA
DATE STARTED 11/13/20 **COMPLETED** 11/13/20
GROUND ELEVATION 117' NAVD 88 **HOLE SIZE** 1.5 sqft
DRILLING CONTRACTOR N/A
GROUND WATER LEVELS:
DRILLING METHOD Hand auger
AT TIME OF DRILLING ---
LOGGED BY RC **CHECKED BY** JG
AT END OF DRILLING ---
NOTES No groundwater or free water seepage observed.
AFTER DRILLING ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT, mixed with sand; dark brown; soft; cohesive, non-plastic; moist. **[Topsoil]**	116.8
0.2		SM		(SM) SILTY SAND, some gravel; ~10-20% fines; orange brown to tan; medium dense to dense; non-cohesive; non-plastic; damp to dry; transitions to weathering rind before refusal on rock. **[Reworked Rock]**	116.0
1.0				SANDSTONE BEDROCK; orange brown to tan; very dense; medium to fine grained. **[Chuckanut Formation (Padden Member)]**	

BORING WAS ADVANCED HORIZONTALLY INTO THE SLOPE.

Bottom of borehole at 1.0 feet.

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BORING NUMBER HA-2

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION 315 Sea Pines Road, Bellingham, WA
DATE STARTED 11/13/20 **COMPLETED** 11/13/20
GROUND ELEVATION 110' NAVD 88 **HOLE SIZE** 1.5 sqft
DRILLING CONTRACTOR N/A
GROUND WATER LEVELS:
DRILLING METHOD Hand auger
AT TIME OF DRILLING ---
LOGGED BY RC **CHECKED BY** JG
AT END OF DRILLING ---
NOTES No groundwater or free water seepage observed.
AFTER DRILLING ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
				0.6	109.4
		SM		(SM) SILTY SAND, some clay, gravel, and occasional cobbles; ~20-40% fines; grayish brown to orange brown; medium dense; low to moderate cohesion; non-plastic; moist to damp; small amount of orange mottling; gravel is sub-rounded to rounded and mostly fine; fines increase with depth. **[Glacial Drift]**	
2.5					
				4.2	105.8

Bottom of borehole at 4.2 feet.

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TEST PIT NUMBER TP-1

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION 315 Sea Pines Road, Bellingham, WA
DATE STARTED 11/13/20 **COMPLETED** 11/13/20
GROUND ELEVATION 115' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Kyle Lukes
GROUND WATER LEVELS:
EXCAVATION METHOD Excavator/back hoe
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES Seepage observed between ~2.5'-3.5' bgs.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	
				1.8	113.2
2.5		SC-SM		(SC-SM) SILTY CLAYEY SAND, some gravel and cobbles; ~30-50% fines; light brown to gray brown, clay lenses are gray with orange mottling; medium dense; cohesive; low-plasticity; moist to wet, seepage observed between approximately 2.5'-3.5' bgs from the north, west, and south pit walls; sand is medium to fine grained; cobbles and gravel are sub-rounded to rounded. **[Glacial Drift]**	
				4.0	111.0
5.0		SM		(SM) SILTY SAND, some fine gravel and cobbles; ~20-40% fines; gray to olive; medium dense to dense; cohesive; non-plastic; damp; small amount of orange mottling; sand is medium to fine grained; gravel is sub-rounded to rounded and mostly fine; bedrock visible at 5.5' bgs on south (downslope) wall of pit; refusal on rock. **[Glacial Drift]**	
				6.8	108.2
				SANDSTONE BEDROCK; dark gray to black; very dense; medium to fine grained. **[Chuckanut Formation (Padden Member)]**	

Bottom of test pit at 6.8 feet.

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TEST PIT NUMBER TP-2

CLIENT Ann C Jones, Family LP
PROJECT NAME Edgemoor Property
PROJECT NUMBER 2020094
PROJECT LOCATION 315 Sea Pines Road, Bellingham, WA
DATE STARTED 11/13/20 **COMPLETED** 11/13/20
GROUND ELEVATION 116' NAVD 88 **TEST PIT SIZE** 15 sqft
EXCAVATION CONTRACTOR Kyle Lukes
GROUND WATER LEVELS:
EXCAVATION METHOD Excavator/back hoe
AT TIME OF EXCAVATION ---
LOGGED BY RC **CHECKED BY** JG
AT END OF EXCAVATION ---
NOTES Seepage observed between ~1.2'-2.5' bgs on N & E pit walls.
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	
0.0					
		OL		(OL) ORGANIC SILT; dark brown; soft; cohesive, non-plastic; moist; root material present. **[Topsoil]**	115.5
0.5					
		SC-SM		(SC-SM) SILTY CLAYEY SAND, some gravel and cobbles; ~30-50% fines; orange brown to gray brown; loose to medium dense; cohesive; low-plasticity; moist to wet, partially saturated in upper region of unit, seepage observed between ~1.2'-3.0' bgs on northern (upland) and eastern pit walls; low to moderate amount of orange mottling throughout; sand is medium to fine; cobbles and gravel are sub-rounded to rounded; some caving around 4'-5' on south (downslope) wall of pit. **[Glacial Drift]**	109.9
6.1					
		SM		(SM) SILTY SAND, some fine gravel and cobbles; ~20-40% fines; gray to olive; dense; cohesive; non-plastic; damp; small amount of orange mottling; gravel is sub-rounded to rounded and mostly fine; cemented and blocky; refusal in hardpan till. **[Glcial Till]**	109.4
6.6					

Bottom of test pit at 6.6 feet.

Exhibit B – Jones Edgemoor Estate - Sea Pines Road Field Photos

Photo 1: TP1

Photo 2: TP1 Location

Photo 3: TP2 Seepage from North Pit-Wall

Photo 4: TP2 Caving on South Pit-Wall

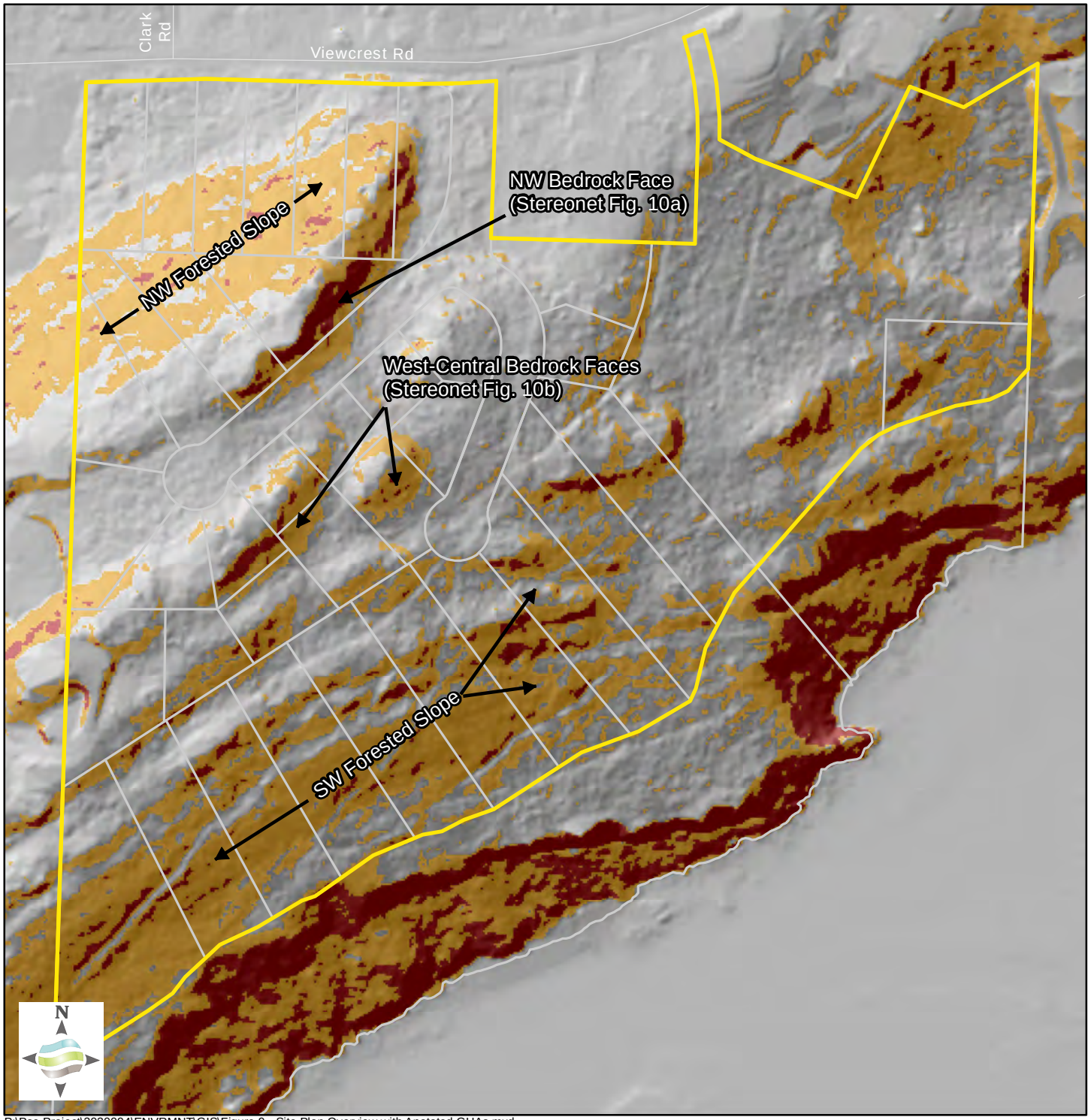
Photo 5: TP2 Location & Site Restoration

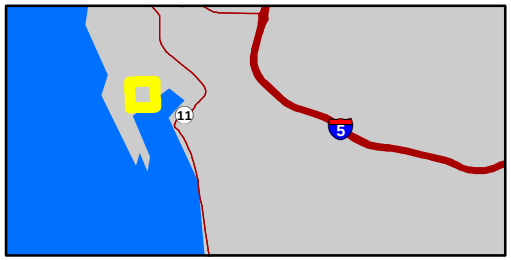
Photo 6: HA-1 Location

Appendix III

1) Figure 9 – Project Overview LiDAR Map with Shading and Geologic Hazard Areas Annotated
2) Exhibit C – Field Photos of Geohazard Slope Features and Rock Exposures
3) Figure 10a – Stereonet of Bedrock Structures – Northwest Hill Cliff Face

   Figure 10b – Stereonet of Bedrock Structures – West-Central Rock Outcrops





Data Credits:
[Parcels] Whatcom County 2018
[Roads] COB 2018
[Lidar] COB 2013

Study Area
Lot Layout
Roads

Percent Slope
0 - 40
40 - 80
80+

Feet
0 250 500

1:2,400

1 inch = 200 feet

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Figure 9

Jones - Edgemoor Estate
Site Plan Overview with
Anotated Geohazards

Date: 2/25/2022

P:\Pse Project\2020094\ENVRMNT\GIS\Figure 9 - Site Plan Overview with Anotated GHAs.mxd

Exhibit C – Jones Edgemoor Estate – Slope/Geohazard Features and Bedrock Conditions

Photo 1: Northern Part of NW Bedrock Face

Photo 2: Southern Part of NW Bedrock Face

Photo 3: Northern Part of NW Bedrock Face

Photo 4: Northern Part of NW Bedrock Face

Photo 5: NW Forested Slope

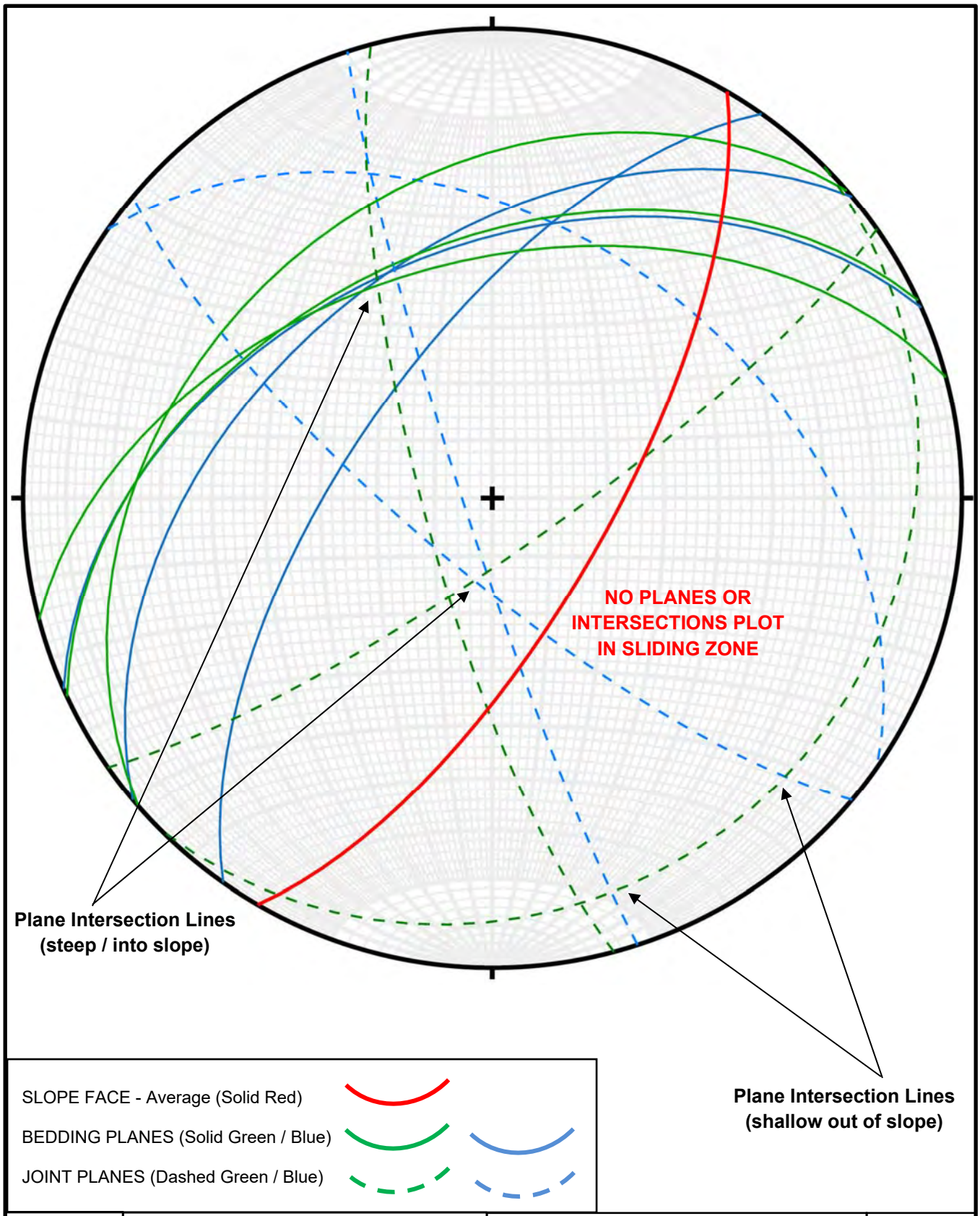
Photo 6: Central Area of NW Bedrock Face

Photo 7: West-Central Bedrock Faces; West Outcrop

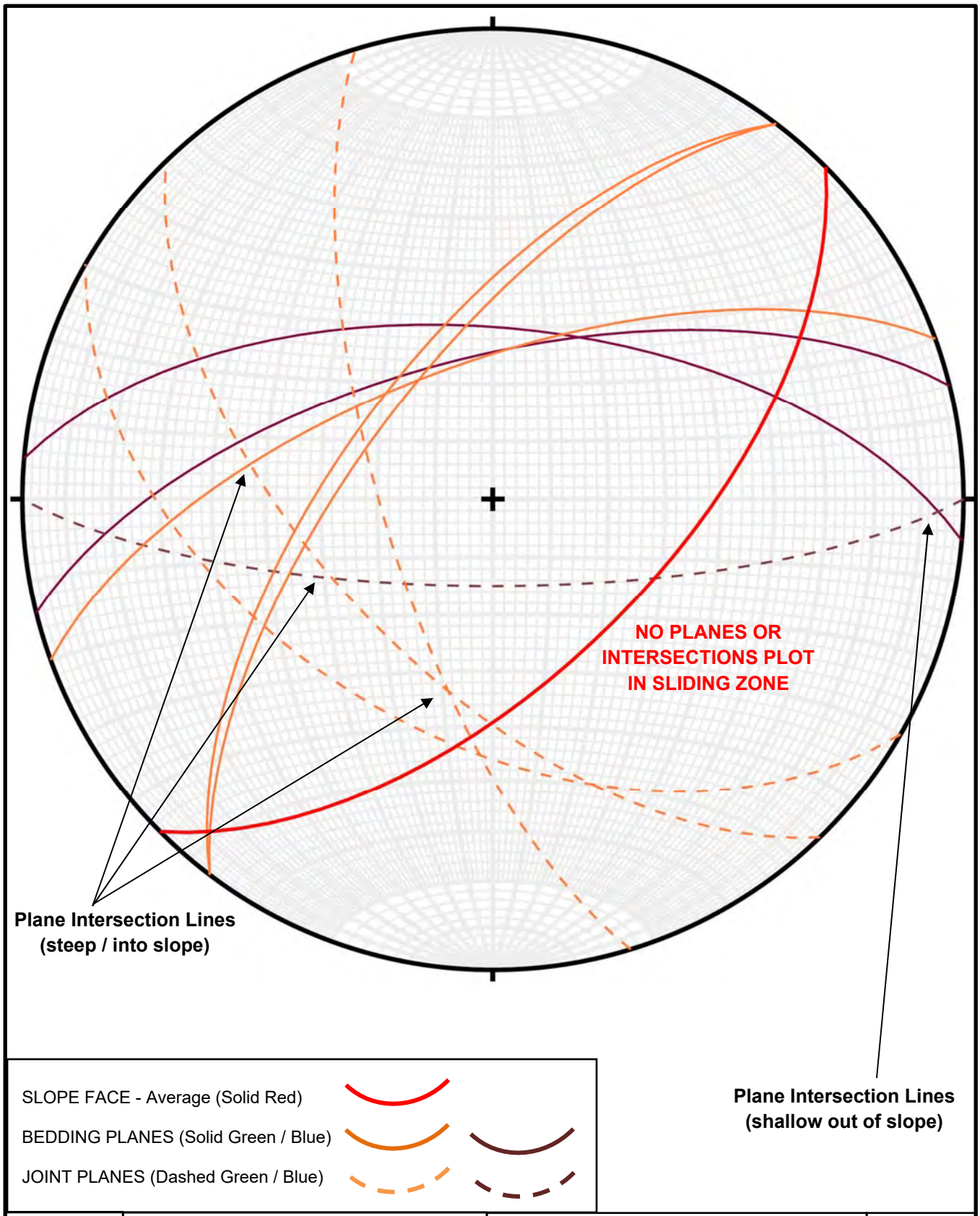
Photo 8: West-Central Bedrock Faces; West Outcrop

Photo 9: West-Central Bedrock Faces; West Outcrop, Northern Slope Area

Photo 10: Conglomerate Bedrock Exposure; SW Project Area



ELEMENT solutions	***NORTHWEST SLOPE SOUTHEAST OUTCROP***	**Stereonet of Bedrock Structures** Jones Edgemoor Estate Geotechnical Investigation & Geohazard	Figure 10a



ELEMENT solutions	***WEST-CENTRAL AREA BEDROCK OUTCROPS***	**Stereonet of Bedrock Structures** Jones Edgemoor Estate Geotechnical Investigation & Geohazard	Figure 10b